LOAD TESTS AND STABILITY OF FOOTINGS AND EMBANKMENTS ON GRANULAR PILE REINFORCED GROUND

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Certificate

This is to certify that the work contained in this thesis, entitled "**Load Tests and Stability of Footings and Embankments on Granular Pile Reinforced Ground,**" has been carried out by *Devendra D. Nagpure* under my supervision and that this work has not been submitted elsewhere for a degree.

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D. D. NAGPURE

Abstract

In the laboratorary, small scale silty soil beds were prepared using a reconsolidation technique. A series of model load tests were carried out on plate with untreated soil, with a single granular pile and with a group of three piles. An attempt is made to correlate results from the load tests on plate with single pile and that with group of piles. Further, stability analyses of footings and embankments constructed on granular pile reinforced ground are performed by using equivalent shear strength concept. Reinforced ground is replaced by an equivalent soil having the same extent. The bearing capacity of the composite ground is determined by an approximate lower bound and a limit equilibrium approaches, assuming the footing to be rigid and the failure surface circular. These approaches predict lower increases in bearing capacity than the observed values. Using modified Bishop's method, stability analyses of embankments on granular pile reinforced ground are performed, assuming circular slip surface. Based on the results of the parametric study, guidlines are suggested for the required extent of treatment of foundation soil below the embankment, for the optimal increase in the factor of safety.

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Notations

a_r		Area replacement ratio
B		Base width of footing or embankment
BCR		Bearing capacity ratio
C_c	:	Consolidation compression index
c_e	:	Cohesion of embankment material
c_{eq}	:	Equivalent cohesion
c_g	:	Cohesion of granular pile material
c_s	:	Geometry constant
c_u	:	Cohesion of soft soil
COV		Coefficient of variation
CU	:	Consolidated undrained triaxial tests
D	:	Depth of soft soil
D_{cr}	:	Depth of the critical slip circle
d_g	:	Diameter of granular pile
d_p	:	Diameter of plate
E_{eq}	:	Equivalent undrained modulus of reinforced soil
E_u	:	Undrained modulus of untreated soil
H	:	Height of the embankment
K	:	constant
LVDT	:	Linear variable displacement transducer
N_1, N_2	:	constants
N_c, N_c^*, N_γ	:	Bearing capacity factors
PI	:	Plasticity Index
Q	:	Load applied
q	:	Applied stress

q_{ult}	:	Ultimate bearing capacity
R	:	Radius of the slip surface
R_{cr}	:	Radius of the critical slip surface
s	:	Center to center distance between granular piles
W	:	Crest width of the embankment
α	:	Treated ground extent factor
γ_e	:	Unit weight of embankment
γ_{eq}	:	Equivalent unit weight
γ_g	:	Unit weight of granular pile material
γ_u	:	Unit weight of soft soil
δ	:	Deformation of th soil
θ	:	Half the angle subtended at the center of trial arc
θ_0	:	Half the angle subtended at the center of the critical arc
ν	:	Poisson's ratio
σ_v	:	Applied stress
σ_{vg}	:	Stress taken by granular pile
σ_{vc}	:	Stress taken by soft soil
σ_1, σ_3	:	Major & Minor principle stresses
ϕ_e	:	Angle of shearing resistance of embankment material
ϕ_{eq}	:	Equivalent angle of shearing resistance
ϕ_g	:	Angle of shearing resistance of granular pile material
ϕ_u	:	Angle of shearing resistance of soft soil

Chapter 1

INTRODUCTION

1.1 Overview

As more and more land becomes subject to urban or industrial developement, good construction sites are difficult to find and hence the use of Ground Improvement Techniques has become the best option, both technically and economically. In many Civil Engineering projects, in-situ ground is generally improved to

- increase the bearing capacity,
- reduce the deformations,
- accelerate the stage of primary consolidation,
- increase the shear strength, and
- reduce susceptibility to liquefaction

The ground improvement methods can be grouped as –

- *Mechanical Modification:* – Soil is densified by application of short term external forces. Surface compaction is carried out by Static, Vibratory/Impact and Plate Rollers whereas deep compaction is performed by heavy tamping at the surface or vibration at depth.
- *Hydraulic Modification.* – In-situ soil is consolidated by lowering ground water table through pumping from boreholes or trenches in coarse grained soils while fine grained soils are improved by preloading with or without sand/plastic drains.

(c) *Physical and Chemical Modification:* - The admixtures like natural soils, industrial by-products, cementitious materials and other chemicals which react with each other and/or with ground are mixed with surface layers or columns of soil at depth to improve the in-situ ground. Grouting and thermal methods heating and freezing are also considered in this category.

(d) *Modification by Inclusion and Confinement.* - Soil is reinforced by fibers, strips bars, meshes, and fabrics (Geosynthetics). Reinforcing elements are used to construct stable earth-retaining structures. In-situ reinforcement is achieved by nails and anchors

The choice of a particular method of ground improvement depends upon many factors such as - the type and degree of improvement required, type of in-situ soil, its geological structure, seepage conditions, cost and type of project, availability of equipment and material, construction time, possible damage to adjacent structures, reliability of methods of analysis and design, etc (Mitchell, 1981 & Hausmann, 1990). However there may be situations in which a combination of two or more of the improvement methods is required for the improvement of the in-situ ground

Among the various ground improvement techniques, granular piles could be the most appropriate choice due to technical feasibility, low energy utilization, cost effectiveness especially in developing countries like India. Large number of case studies also indicate that many difficult foundation sites throughout the world have been and are being improved by installation of granular piles. Besides improving strength and deformation properties, they densify in situ soil, drain rapidly the generated pore pressures, accelerate consolidation and minimize post installation settlements. Granular piles allow the treatment of a wide range of soils, ranging from loose sands to clays by forming reinforcing elements of low compressibility and high shear strength. Granular piles can be installed by Vibro-compaction, Vibro-replacement, Vibro-Compozer, Cased Borehole methods (rammed stone columns) and even by heavy tamping depending upon their proven applicability and availability of equipment in the locality. The rammed stone columns incorporate the additional benefits of heavy tamping as they in effect are preloaded

Granular piles increase the resistance to liquefaction and minimize settlements following it (Baez & Martin, 1992). No damage was observed from the treated sites wherein granular piles were used to improve the site characteristics which were subjected to recent Loma Prieta Earthquake (Mithchell & Wentz, 1991). Granular piles mitigate the

potential for liquefaction and damage by (i) preventing build up high pore pressure (ii) providing drainage path and (iii) increasing the strength and stiffness of ground Saito *et al* (1987) recommended that stone sizes be selected to satisfy the filter criteria $20\, d_{s15} < d_{c15} < 9\, d_{s85}$, where d_{s15} and d_{s85} correspond to 15% and 85% finer sizes of in-situ soils and d_{c15} to the stone column material

Reported possible failure mechanisms for single granular pile are - bulging, general shear and pile failure. However Madhav & Miura (1994) report that bulging and pile failure are not mutually exclusive Based on Van Impe & de-Beer (1983) approach, Van Impe & Madhav (1992) show that significant reduction in settlement can be achieved by considering the dilatancy of the granular pile material

In the following section, technical literature dealing with granular piles has been reviewed The scope of literature has been decided to reflect the scope of further research required in the areas discussed in the thesis.

1.2 Literature Review

In this section a brief review pertinent to areas related with granular piles that have been proposed to be studied in this thesis are presented

1.2.1 General

One of the oldest historical example of the use of granular piles is found in 1830's by French military engineers, but the modern origin truly began in 1930's in Germany by their Russian Emigre (Alamgir *et al.*, 1994) The theoretical background, analysis, design aspects and installation techniques have been developed by various researchers following which this method has been used extensively for site improvement. Some of these works have been reviewed and discussed at several conferences and symposia by Mitchell (1981), Madhav (1982), Barksdale & Bachus (1983), Aboshi & Suematsu (1985), Ranjan (1989) and Bergado *et al* (1991).

1.2.2 Load Tests

Engelhardt & Golding (1975) presented a large scale field testing data pertaining to the stone column improved ground for construction of a sewage treatment plant on deep,

soft, cohesive soils in area of high seismic suceptibility. Field tests were conducted to demonstrate the densification of sand lenses in cohesive sub-soil with respect to liquefaction potential, the improvement in strength properties to resist the horizontal forces and load-settlement relationship.

Hughes *et al.* (1975) proposed a method to predict the load-settlement relationship for an isolated stone column in soft clay prior to field testing. The radial stress strain data obtained from Cambridge pressuremeter was used to predict the load-settlement relationship. It has been concluded that the estimation is reasonable if allowance is made for the load transfer from column to in-situ soil through side shear and determination of column size accurately.

Nayak (1982) proposed a rational approach for the design of stone columns. He deviated from general practice of conducting plate load tests directly on a single stone column to test the area represented by the single stone column. It was observed that the settlement under actual foundation will not exceed 3 times that from load test on a single stone column.

Barksdale & Bachus (1983) in their State-of-the-Art report presented a guide specification for performing vertical, short term (undrained) load tests. Based on small model test studies in soft clay, they report that the method of applying load influences the ultimate capacity of a stone column. They suggested that the load should be applied through rigid plate or footing having area larger than the area of stone column. Further they also discussed the case histories in which vertical load tests and one lateral load test (direct shear test) were performed.

Based on full scale load tests on embankments constructed over granular piles, Bergado *et al.* (1984) have reported that the granular piles reduce the settlement of soft clay foundation by as much as 20 to 40 % as compared to prefabricated vertical drains. They also found that the ultimate pile bearing capacity was 3 to 4 times greater than that of untreated ground and the piles acted independently when spacing was 3 times pile diameter or greater.

Bergado & Lam (1987) investigated the behaviour of granular piles with different densities and different proportions of gravel and sand on soft Bangkok clay, by performing full scale load tests. Three groups with 3 pile each, and two groups with two piles each were installed using a non-displacement cased borehole method. It was observed that for the same granular pile material, the ultimate pile capacity increases with increase in

the number of blows per layer and gravel is the most efficient granular pile material with high friction angle at smaller number of blows per layer. They reported that the ultimate pile capacities and the load-settlement curves predicted from the method of Hughes *et al* (1975) were in good agreement with experimental data. Similar to results of Hughes *et al* (1975), they observed the bulging of granular pile to occur between one third to one pile diameter below ground surface.

Datye & Madhav (1988) studied the performance of foundations on stone column improved ground. A wide range of case histories were studied. A simple linear elastic model of stone column behaviour and equilibrium relations were used with success to predict the behaviour of improved ground. The need for the semi-empirical methods and use of load tests to evaluate the parameters was underlined in view of probable disturbance caused during stone column installation and the difficulty in precise theoretical assessment of the consequence of the stress changes during installation.

Using small scale model in soft soil, Madhav & Thiruselvam (1988) have studied the effects of the method of installation — cased or uncased holes, number of lifts, compaction energy given per lift given to granular pile and pile spacing. They concluded that the settlements are less in case of cased bore holes than in case of uncased bore holes. Larger the compactive energy, the more number of lifts and closer is the spacing, the better is the response of granular piles.

DeStephen *et al.* (1993) presented a case history in which floating stone columns were used to support a Processing Center at a Nuclear Power plant. Short stone columns were installed into deep hydraulic fill which was placed to reclaim the power station area. They have described a vertical load test performed on group of three columns using square plate, prior to installing production stone columns. The load-settlement relationship was observed to be almost linear upto the twice design bearing pressure. As dry installation technique with bottom feed system was used, heave of ground of about 15 *cm* was observed during the construction of stone columns.

Leung & Tan (1993) conducted a laboratory investigation to study the load distribution and consolidation characteristics of a composite soil reinforced by a sand column. Under a constant applied pressure, they observed that the sand columns progressively carried more loads and reached a maximum stress concentration factor after consolidation of clay has been completed. The maximum stress concentration ratio increased with increasing area replacement ratio and appeared to be independent of the surcharge load-

ing pressure. They further reported that the magnitude of load carried by sand columns depends upon the stiffness of sand column material and stated the need for further studies to investigate the various factors that may affect the load distribution in composite soil

Ketkar & Telang (1991) discussed the relative performance of rammed stone piles vibro stone columns and sand drains (sandwicks) used to improve ground consisting of very soft marine clay for the construction of petroleum storage tanks Ground treated with rammed stone piles resulted in 14% more stone consumption and deformed 15% less (both under short term load tests and long term hydrotesting) than those for the vibro stone columns However the time for construction of rammed stone columns was reported to be 6 3 times higher than that for vibro stone columns. Sandwicks took the least time for construction but longer time for hydrotesting and lead to very high settlements, same as for the untreated ground

Lee (1994) presented a case history in which foundation soil is improved by vibroflotation to reduce the deformations for construction of two power and desalination plants He pointed out the importance of continuous sounding for the selection of appropriate soil improvement method The foundation soil consisting of sand with traces of silt or silty sand was improved using the improvement procedure determined from the result of field trial program, together with quality control criterion Based on field results, he concluded that cone penetration test is an adequate test for quality control work in cohesionless soil while plate load test is good alternative for cohesive soils Soil treated with stone backfill resulted in better performance for controlling settlements than that treated using sand backfill.

1.2.3 Bearing Capacity

Madhav & Vitkar (1978) proposed a method to estimate the ultimate bearing capacity of a strip footing constructed on soils stabilized with granular trench They have postulated a failure mechanism for such foundations and have derived analytical expressions for the ultimate bearing capacity using upper bound theorem of limit analysis. From this study they have affirmed that a granular trench significantly reinforces weak clay deposits.

Based on analytical studies, Madhav (1989) proposed the methods of predicting bearing capacity (considering both bulging and pile failure mechanisms) and deformations of granular pile-raft system. The efficiency of this system in increasing bearing capacity of

the raft was also demonstrated. The predicted values of bearing capacity ratio for pile failures were found to be close to the measured values from small scale in-situ tests.

By adopting different limit equilibrium methods of analyses Enoki *et al.* (1991) made a comparative study to predict bearing capacities for ground having homogeneous clayey, heterogeneous clayey and sandy soil. Fellenius method has been compared with other approaches and found that for clayey ground, Fellenius method overestimates the bearing capacity but underestimates the same for ground having sandy soil. They presented an approach to determine the shear strength of composite ground reinforced with sand piles, in terms of equivalent soil parameters and performed bearing capacity analysis for improved ground. They found that their analysis using equivalent soil parameters overestimates bearing capacity for low area ratios and underestimates for higher area ratios when compared with conventional approach using equivalent soil parameters and the method suggested by Aboshi *et al.* (1979).

Asaoka *et al* (1994) determined the bearing capacity of soft clay improved with sand compaction piles based on soil-water coupling limit analysis, using Cam Clay model. They found that for rigid-rough footing, drained condition provides greater bearing capacity than undrained condition for sand due to stress concentration at the top of sand piles. However, embankments (flexible loading) yield greater bearing capacity than undrained condition due to dilatancy characteristics of sand. Based on field loading test, they reported that bearing capacity of soft clay improved with sand compaction piles increases due to consolidation of surrounding clay caused by sand pile driven into clay irrespective of disturbance of soft clay.

Bouassida *et al* (1995) predicted bearing capacity of soft soil reinforced by a group of columns (stone columns and lime columns) using yield design theory. Similar to equivalent soil parameters given by Enoki *et al* (1991), they have derived expressions to determine the equivalent soil parameters. It is interesting to note that for a given area ratio, with increase in the value of angle of shearing resistance of column material, ϕ_c, the equivalent cohesion reduces for values of ϕ_c upto $27° - 30°$. However with further increase in ϕ_c, equivalent cohesion is found to increase rather than decrease. Also the expression given for equivalent cohesion does not satisfy the end condition for the case of fully reinforced ground i. e. for $a_r = 1.0$. This may be due to their assumption made regarding the estimation of lateral stress acting on the column. Their approach appears to be appropriate for improvement in bearing capacity of soft soil using lime columns whereas for stone

columns, it yields very low improvement

1.2.4 Shear Resistance

Rathgeb & Kutzner (1975) have described case history in which the foundation soil was improved by the vibroreplacement process to obtain increased bearing capacity and to reduce settlements. They have carried out the stability analysis of a embankment for a motorway constructed over soft ground in which only the area under the embankment side slope was treated by granular piles. They used circular slip surface analysis based on Fellenius method and grid pattern search technique to obtain the minimum factor of safety.

Priebe (1976) suggested a theoretical approach using the concept of infinite grid of stone columns beneath a rigid raft. A constant volume for the compacted backfill material was assumed and the influence of the dead weight of the soil and columns was neglected. Using equations of elasticity and Rankine earth pressure theory, equations were developed for the settlement behaviour and shearing resistance of the improved ground. Charts were presented for rapid determination of equivalent soil parameters used in stability analysis for embankments constructed on improved ground using granular piles.

Almedia *et al* (1985) conducted centrifuge tests at 100-g, of embankments constructed on strengthened, normally consolidated kaolin clay. Tests were performed using stage construction techniques for average embankment heights of 11.6 m and 13 m for unreinforced and reinforced cases respectively. Effective stress analysis produced factors of safety below unity, which they claimed is a consequence of the side friction developed inside the centrifuge model container. They further suggested that for embankments on granular piles, small factor of safety of the order of 1.1 to 1.2 can be tolerated at the start of each stage of construction.

The shearing resistance of the soft ground reinforced by granular piles is estimated based on planer shear failure mechanism. Madhav (1992) carried out direct shear tests on model unit cell and found that the shear resistance of the composite granular pile reinforced ground depends upon the stress level, the modular ratio of granular pile and soft soil and on the inclination of the failure surface, in addition to area ratio. He also added that the modular ratio, actually operative in the field could be somewhat larger than the values estimated based on 1-dimensional tests.

1.3 Scope of & Motivation for the Thesis

In the previous section, a brief review of literature pertaining to the field and laboratory load tests on granular piles bearing capacity of composite ground and stability of embankments constructed on soft ground reinforced with granular piles is presented The motivation for further research in the above areas and organization of thesis work are presented as follows

Review pertaining to load tests performed on single granular pile or group of piles indicate that field load testing is an important stage to determine the quality control criterion and to verify the design specifications for the effective performance of the improved ground Effects of various parameters that may affect the performance of granular piles were studied by both full scale (in-situ) and small model laboratory load tests. It is usual practice to conduct load test either on single granular pile or on group of granular piles (in most cases three piles) This indicates that it is necessary to determine the correlation between the load test on single pile and that on group of piles so that the relation can be extrapolated to predict the performance of composite ground obtained by installing the granular piles.

Literature review pertaining to bearing capacity indicates that several design approaches are available to determine the ultimate bearing capacity of composite ground reinforced with single granular pile based on different modes of failure. These approaches assume a trench like reinforcement under the plane strain assumption (Madhav & Vitkar 1978) and for an isolated column in axisymmetric condition. Large number of field tests indicate the validity of these approaches. Very few attempts (Enoki *et al.*, 1991 and Bouassida *et al.*, 1995) have been made to deal with the case of a soil reinforced by group of granular piles However these approaches underestimate the bearing capacity of reinforced ground compared to the actual observed values. Hence it is necessary to develope a method which can predict appropriate results and considers the effect of failure surface

Literature review pertaining to stability analysis of reinforced embankments on reinforced soft soil with granular piles reveals that different approaches based on limit equilibrium method are currently available for the stability analysis of the embankments Generally the stability analysis is performed assuming the critical surface as circular slip surface. In one of the earliest work, Rathgeb & Kutzner (1975) analyzed the stability of embankments constructed on soft ground reinforced with granular piles using the Fellenius method assuming circular slip surface The shear parameters of composite soil are

determined by using the approaches suggested by Priebe (1978), Barksdale & Bachus (1983) and Enoki *et al* (1991) Aboshi *et al* (1979) presented a method which takes into account the effect of stress concentration. Sabhahit (1994) introduced the concept of the optimum pile length for the maximum increase in factor of safety for a given embankment From these studies it is found that there is a need to develope the guidelines regarding the extent of treatment of foundation soil for the optimal increase in factor of safety

Based on above discussion, the topics for the present investigation has been chosen The scope and organization of the thesis is described below

In Chapter 2, the procedure to prepare soft clay bed using reconsolidation technique is explained The small scale model load tests conducted on composite ground reinforced with single granular pile or group of piles are described

The results obtained from these model tests are analyzed in Chapter 3. Discussions and conclusions are made based on the test results

The procedure to determine bearing capacity for reinforced ground with group of piles assuming rigid footing is developed in the Chapter 4. Effect of different variables on bearing capacity is studied.

Chapter 5 deals with stability of embankments constructed on soft ground reinforced with granular piles Using equivalent shear strength parameters and assuming circular slip surfaces, stability analyses are carried out. Based on the results of stability analysis, guidelines are suggested for the required extent of treatment of foundation soil for the optimal increase in factor of safety

Results and conclusions obtained from these studies are presented in the Chapter 6 The scope for further investigation is mentioned.

Chapter 2

LOAD TESTS

2.1 General

To develop a quality control program or to verify overall design specifications for granular pile reinforced ground, standard penetration tests (SPT), cone penetration tests (CPT) and/or plate load tests are performed, depending upon the type of sub soil. Installation of granular piles in cohesive soil results in the deposition of particles in more dense state whereas that gives the reinforcement effect to cohesive subsoil. For ground involving cohesionless soil, CPTs are particularly useful for quality control work which are normally performed in the center of the grid to be on a conservative side (Lee, 1994). Plate load tests are useful for evaluation of effectiveness of granular piles in cohesive subsoil Plate load tests are carried out to determine (i) ultimate bearing capacity, (ii) settlement characteristics, (iii) shear strength of the composite ground, and (iv) to verify the adequacy of the overall construction process The type and number of field tests performed depend upon the specific application of the granular piles and also many other factors such as subsurface conditions and the degree of conservatism used in design.

From a literature review it is observed that the majority of reported field tests are vertical, short-term (undrained) load tests, performed on either a single granular pile or on a group of granular piles (mostly three piles) In the present analysis, an attempt is made to correlate the results obtained from the plate load tests on untreated soil, a single pile and on a group of three piles. In the laboratory, silty soil beds are prepared by using *reconsolidation technique*, as suggested by McManus & Kulhawy (1993) Granular piles are constructed and small scale model load tests carried out

2.2 Preparation of Soil Deposits

The earlier studies indicate that the model load tests were conducted by using either small scale in-situ soil deposits (Madhav 1992) or remoulded soil samples (Hughes & Withers, 1975, Madhav & Thirusevlem, 1988, Luang & Tan, 1992) In the present load testing program, soil beds are prepared by using the reconsolidation technique. McManus & Kulhawy (1993) describe the detailed methodology of this reconsolidation technique to produce uniform and identical soil deposits in the laboratory.

Even though no soil in nature is formed by the process of reconsolidation, this method is *adopted due to the following reasons*

- This method removes many uncertainties with field deposits, including in-situ stresses stress history and soil inhomogeneity
- Soil properties are reproducible

2.2.1 Principle of Preparation

Slurry was prepared and poured into the tank above a sand layer Vertical surcharge stress was applied through a mechanical jack at the top surface as shown in Fig. 2 1. Filtered side drains and the bottom sand layer discharged excess pore water at atmospheric pressure

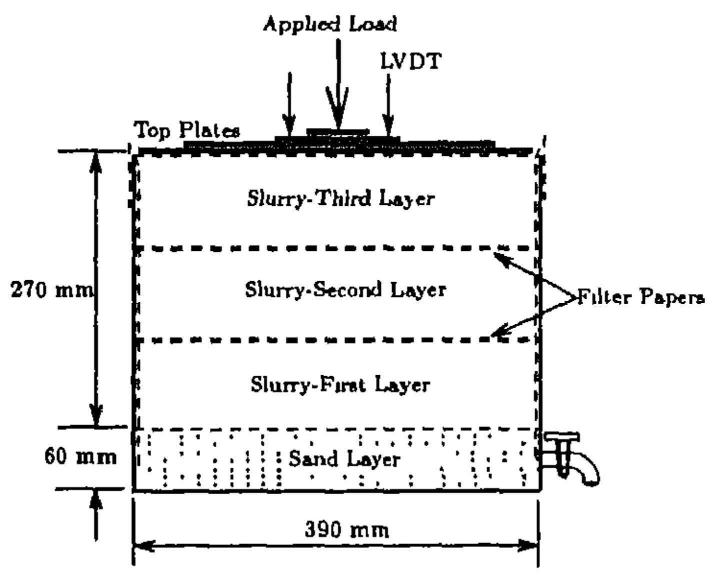

Figure 2.1. *Schematic of Consolidation System*

2.2.2 Soil Used

Locally available campus silty soil is used for the preparation of the soil beds This soil is an alluvial deposit and has composition of 10 12% sand, 75 80% silt and 10-15% clay The liquid limit, w_L and plasticity index, PI of the silt were 31% and 15% respectively I S Classification of this soil is - CL

2.2.3 Slurry Preparation

Silty soil was air dried, broken down to powder form and then used for the preparation of slurry Required quantity of soil was mixed with known amount of water in a tub and the mixture was kept for about a week for saturation The normal practice of making slurry for preparation of soil beds using the reconsolidation technique, is at a initial water content equal to twice of the liquid limit However in the present experimental study hand-mixed slurry was prepared at the initial water content varying between 40 and 12% i e 1 17–1 24 times the liquid limit This criteria was selected to avoid very large volume change taking place during the consolidation phase and thereby obtaining deposits having moderate thicknesses.

2.2.4 Consolidation Tanks

Two tanks made of GI sheets, each 390 mm diameter by 330 mm deep, were constructed The size of tanks was selected to keep the weight manageable during handling of the tank For the lateral rigidity of the tank, four small metallic strips having width 30 mm were attached at the outer periphery of the tank To allow drainage of excess pore water, a one–way valve was provided near the base of the tank. The experimental setup is shown in the Fig. 2.2.

2.2.5 Sample Preparation

A 60 mm layer of Ganga sand was placed at the bottom of the tank. It was kept saturated with water for a period of about 10–12 hours. The excess water was allowed to drain through the bottom valve. The sand was covered with a circular filter paper to avoid intermixing of the sand and slurry Side filter drains in the form of strips were placed along the inner surface of the tank Slurry was well mixed to ensure that mixture was

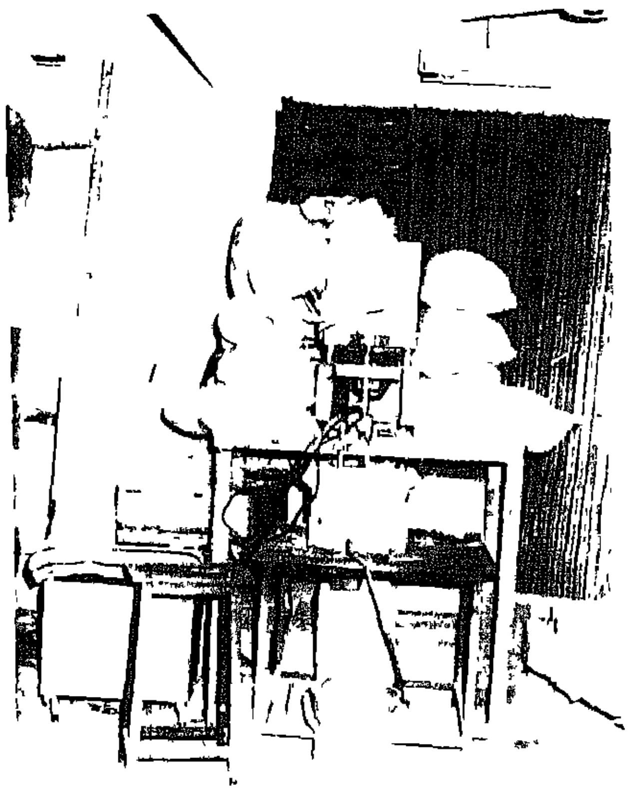

Figure 2.2: *View of Experimental Setup*

homogeneous. Then first layer of slurry was poured into the tank, upto 90 mm (one-third height of the tank). The top surface was leveled off and two layers of filter paper in the form of sectors were placed. This procedure was repeated for the second layer of the slurry. The topmost surface of the third slurry layer was covered with a circular filter paper. Then at the top, a metallic plate having diameter 380 mm was placed. In order to distribute the applied stress uniformly over the tank, three smaller plates of diameters 300, 125 and 57 mm were used.

2.2.6 Loading

Vertical surcharge stress was applied in two stages. In the first stage, only top four metallic plates were used resulting in a very low consolidation pressure of 0.5 kPa. This was selected to avoid squeezing of the slurry. During the first phase, surface settlement

was measured with a mechanical dial guage placed at the center of top plate The dial guage has a least count of 0 01 mm and 50 mm travel This was kept for a period of 24 hours Then the dial guage was removed and tank was shifted and centered below the loading frame

In the second phase, vertical stress was applied through a mechanical, hand-operated jack Deadweight supported on metallic frame as shown in the Fig 2.2, was used to provide the reaction for the mechanical jack Load was applied vertically and concentric to the tank A ball bearing arrangement was used to avoid any possible eccentricity of the loading Applied load was measured with an electronic load cell having least count of 0 01 kN and total capacity of 10 kN Observations of vertical displacement of the top surface were made with three linear variable displacement transducers (LVDTs), having 20 mm travel and least count of 0.01 mm Plastic cylinders having 25 mm height were provided to obtain the continuity in extending the range of the guages Two of LVDTs were positioned diametrically opposite to each other, at equal distances from the center of the tank, the third is at right angles to the others LVDTs were attached to cross-channels of the loading frame by metallic clamps

Load was increased progressively to the maximum pressure (22 kPa) in a period of nine hours resulting in rate of loading about 2 45 kPa per hour The maximum pressure was continuously monitored during next four days. It was ensured that the top surface was kept flooded with water On the sixth day, unloading was done in four hours The typical load settlement curve during the consolidation process is shown in Fig 2 3

After unloading, tank was shifted and the top plates were removed Then granular piles were installed

2.3 Installation of Granular Piles

Granular pile construction involves partial replacement of subsoil with a compacted vertical column of granular material that usually completely penetrates the whole weak strata. In the present study, a method similar to the construction of *rammed stone columns* is used.

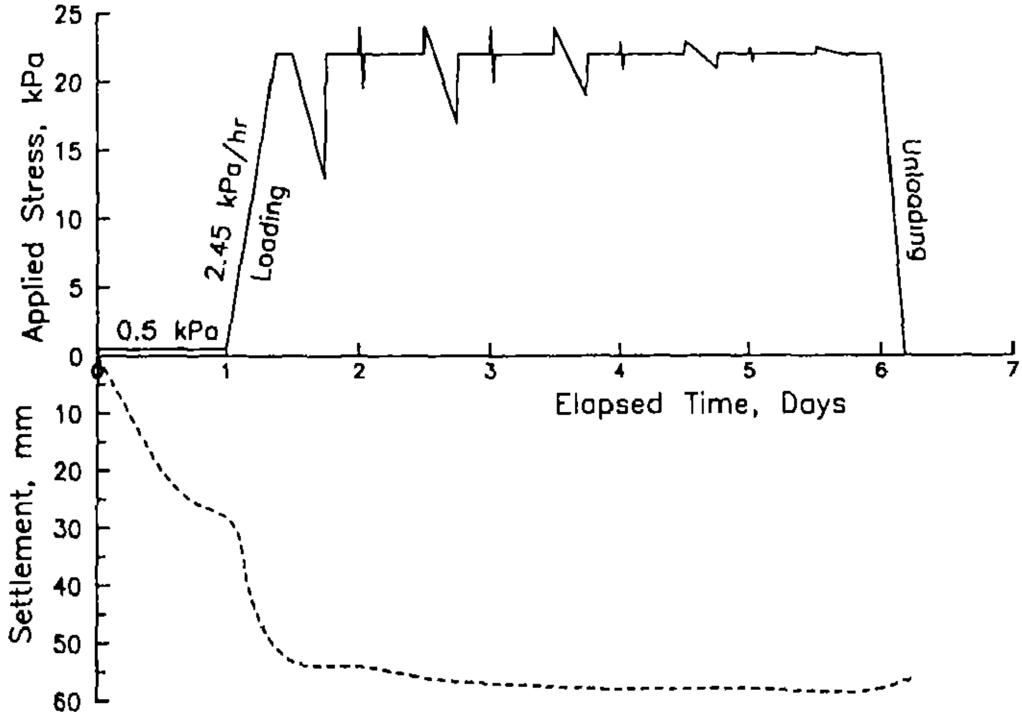

Figure 2.3 *Load-Settlement during Consolidation of Deposit D3*

2.3.1 Granular Pile Material

The granular pile material used to construct these piles consists of a mixture of gravel and Kalpi sand in 2:1 proportion, by mass. The gravel material is whitish-gray limestone crushed aggregates. Kalpi sand is not a typical river sand like Ganga sand. It is bulky having elongated, angular grains coated with carbonate. The grain size distribution curves of these materials are shown in the Fig 2.4.

Based on the results of the sieve analysis, the sand, the gravel and the gravel-sand mixture were classified, respectively, as well graded sand, poorly graded gravel and gap graded sandy gravel. The minimum and maximum unit weights of the gravel-sand mixture were 15.961 and 18.462 kN/m^3 respectively. Oedometer tests indicated the compression index C_c = 0.012 to 0.014 and coefficient of compressibility, m_v = 1.3885 to 0.185 m^2/MN. To obtain the internal friction angle strain controlled laboratory direct shear tests were performed. The angle of shearing resistance of granular material, ϕ_g ranged from 38^0 to 43^0.

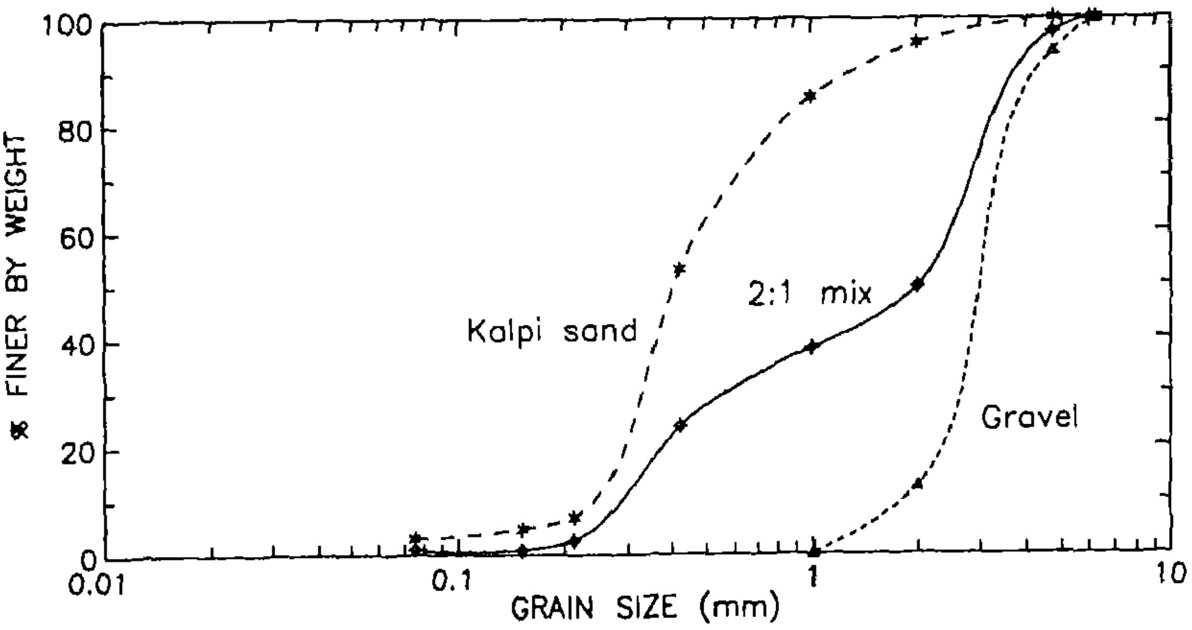

Figure 2.4. *Grain Size Distribution of Granular Materials*

2.3.2 Installation Procedure

A hole of required diameter was made by driving an open-ended pipe and then extracting the soil by rotating and subsequent withdrawal of the pipe The uncased hole was filled with the gravel-sand mixture, in three layers and each layer was tamped uniformly After installation of granular piles in a deposit, the top surface was covered with a thin layer of Kalpi sand

2.4 Load Tests

A series of small scale, model plate load tests were carried out. The load from the loading frame was applied through rigid circular plates of diameter 40, 75 and 125 mm. Loads were increased in stages and settlements were measured through LVDTs with a least count of 0 01 mm. Readings for each load increment were taken till the change in the settlement was less than 0.02 mm in 1 min. Test was continued till either a stress level of 200 kPa or a total settlement of 20 cm was reached

Three plate load tests were performed on a single soil deposit. The typical test arrangement is shown in Fig. 2.5.

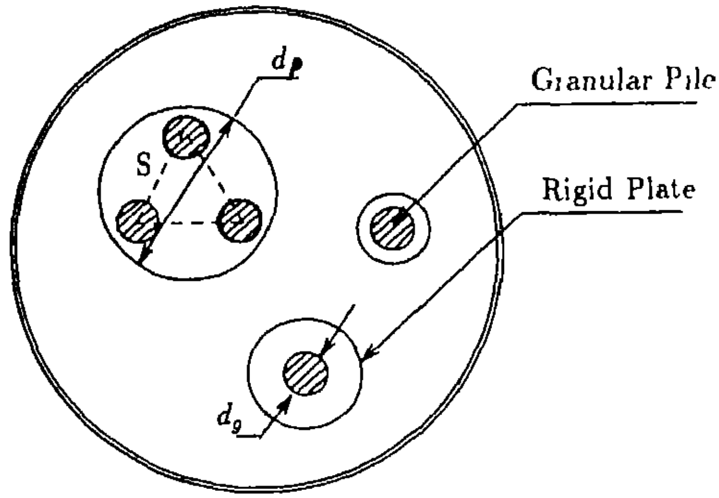

Figure 2.5 *Typical Load Test Arrangement*

The parameters varied were the number of granular piles (no pile, a single pile and a group of three piles), diameter of granular pile (d_g = 25, 30 and 33), diameter of plate (d_p = 40, 75 and 125) and spacing of granular piles (s/d_g ratio = 2, 2.5 and 3). The results obtained from these load tests are presented in the next Chapter.

Chapter 3

DATA INTERPRETATION & DISCUSSION

3.1 General

In this Chapter, the results obtained from the reconsolidation of the soil and the plate load tests are presented and analyzed. Based on the analysis, discussion and conclusions are made.

3.2 Consolidation Results

During consolidation, vertical stress and top surface displacement were monitored and recorded continuously. The surface displacement measurements were used to estimate the end of primary consolidation. Pore water stress measurements could not be made.

3.2.1 Surface Settlement

The average surface settlement during the consolidation process is observed to be about 55–60 mm (20–22% of the total initial height of the slurry sample). The reported settlements (Mcmanus & Kulhawy, 1993) during the reconsolidation process are about 50% of the initial height of the slurry volume with the initial water content equal to twice the liquid limit, w_L of the soil sample. In the present study, slurry was prepared at the initial water content equal to about 1.2 times w_L.

In some of the deposits, differential settlements of about 1 2 cm were observed. This may be due to the eccentricity of the load applied during the consolidation process. The typical time-settlement curve during the consolidation process is depicted in Fig 2 3

3.2.2 Consolidation Time

Consolidation was considered to be essentially complete when there was no further measurable change in the top plate displacement. The amount of time required to consolidate the soil deposit estimated was about 1.78 to 3.30 days. This is based on the values of coefficient of consolidation, $c_v = 1.18\times10^{-7}$ to 0.64×10^{-7} m^2/s, obtained from remoulded sample at initial water content of 25%. McManus & Kulhawy (1993) suggested the time required for the preparation of such deposits may be calculated as depth in meter times 15 days. Using this prediction, the consolidation time is about 1.35 days. Oedometer tests performed on the samples taken from the reconsolidated deposits indicated c_v = 2.93×10^{-7} to 1.35×10^{-7} m^2/s which resulted in the consolidation time of about 0.72 to 1.36 days. *Hence the predictions based on the remoulded sample overestimated the consolidation time actually observed. McManus & Kulhawy's criteria is found to be in the observed time range.* However the loading was continued till there were no significant fluctuations in the applied stress intensity. This resulted in total time span of 5 days.

3.2.3 Consolidation Indices

The oedometer tests performed on the reconsolidated samples indicated the compression index C_c = 0.113 and C_c of the granular pile material (reported in Section 2.3.1) is 0.013. The ratio of compression indices of the granular pile material and the reconsolidated soil is about 9. The compression ratio of reconsolidated sample, $\frac{C_c}{1+e_0}$ is equal to 0.1794. The coefficient of volume change, m_v = 1.3885 to 0.185 m^2/MN, for the stress range of 25 to 200 kPa. The maximum past consolidation stress, $\overline{\sigma}_{vm}$ is observed to be between 10 to 15 kPa, indicating that all the stress applied (22 kPa) was not virgin stress.

3.2.4 Water Content Profile

Large number of water content measurements were made by dissecting the deposits and during the excavations for installation of granular piles. The mean coefficient of variation

(COV) for individual deposits was typically 0.01 to 0.02, with a worst case of 0.05, indicating that a high degree of uniformity was achieved within each deposit. The mean water content values of the individual soil deposits ranged from 24.1 to 26.7% with an overall mean value of 25.3 and a COV of 2% indicating that good repeatability was achieved from one deposit to another. The variation of water content verses depth is plotted in Fig. 3.1 which shows a steady decrease in water content with increasing depth, as expected. The

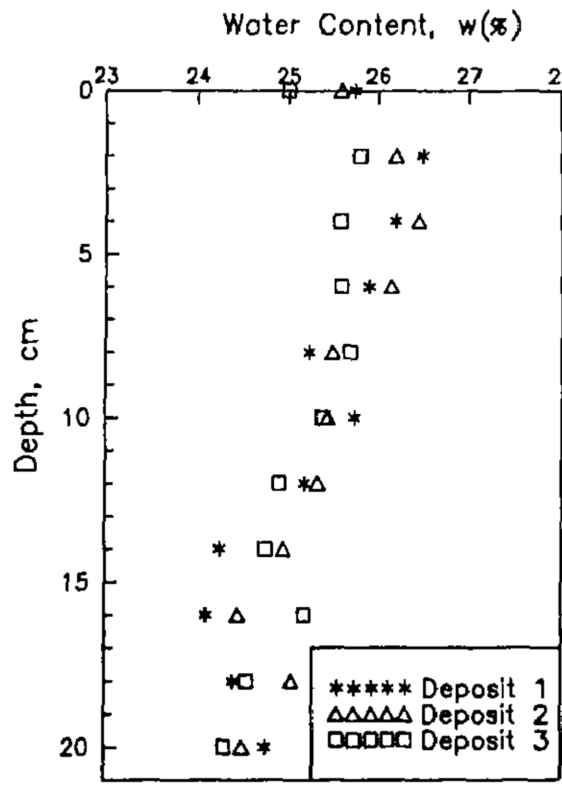

Figure 3.1 *Water content verses Depth of the Deposits* D1, D2 & D3

trend in variation in the water content is small, and the uniformity throughout the deposit appears to be high.

3.2.5 Subsurface Settlement

During dissection of the soil deposit, inner settlement profile was obtained from the positions of filter papers placed inside the tank. The differential settlement of about 2 cm was observed, almost in all of the deposits. This is possibly due to the side friction developed along the inner surface of the tank.

3.2.6 Strength Measurement

Shear strength measurements were made by performing the fall cone tests. The apparatus used is similar to the penetrometer specified by IS 2720 (Part V) 1985 having a stainless steel cone with 30° apex angle and 100 g mass The procedure and the correction factor are as suggested by Wood (1985). The undrained shear strength c_u ranged between 6 9 and 10 3 kPa Vane shear tests could not be performed as difficulties occurred while determining the spring correction factor. Further, the available Pocket Penetrometer was found unsuitable as it has least count of 25 kPa Triaxial UU tests resulted in $c_u = 10$ kPa and $\phi_u = 4^o$

3.3 Post-Installation Observations

It was observed that during installation of granular piles, the diameter of the granular pile is slightly greater than the pipe diameter. This is due to enlargement of uncased holes during tamping of the granular material The observed diameters of the piles were 27, 33 and 36 mm

Similar to a case study reported by DeStephen *et al* (1992), surface heave of about 0.5 to 1 0 cm was observed This is because the dry method displaces the surrounding soil during the installation of granular pile.

3.4 Load Test Results

As mentioned earlier, three types of small scale, model load tests were performed on plate with–unreinforced ground (UPLTs), a single granular pile (SPLTs) and a group of three granular piles (GPLTs) The parameters varied are the diameters of the plate and of the granular pile and spacing of the piles The details of these load tests are summerized in Table 3 1.

In Fig. 3.2, the typical load–settlement, time–settlement and stress–settlement curves are presented for the load test SPLT5. It can be observed from this figure that the total settlement is composed of immediate settlement upon loading and consolidation settlement from the silty soil. A single unloading stage was also conducted for this test. The subsoil below the plate exhibited permanent deformation after unloading This appears

due to the consolidation of the silty soil The remaining load tests were performed without the unloading stage

Table 3.1: *Summary of Load Tests*

Load Test No	*Type of Test*	*Notation*	*Dia of Plate* d_p, mm	*Dia of Gr Pile* d_g, mm	*Pile Spacing* S, mm	*Surface Area Ratio* η %
1	Unreinforced	UPLT1	40	–	-	-
2		UPLT2	75	–	–	-
3		UPLT3	125	–	–	
4	Single Pile	SPLT1	40	36	–	81.00%
5		SPLT2		33	-	68.06%
6		SPLT3		27	–	45 56%
7		SPLT4	75	36	–	23.04%
8		SPLT5		33	–	19 36%
9		SPLT6		27	–	12.96%
10		SPLT7	125	36	–	8 30%
11		SPLT8		33	–	6 97%
12		SPLT9		27	–	4.67%
13.	Group of Piles	GPLT1	125	27	55	14.00%
14.		GPLT2			68	14 00%
15		GPLT3			81	14.00%
16.		GPLT4		33	66	20.91%
17.		GPLT5			75	20.91%
18.		GPLT6			99	20.60%
19.		GPLT7		36	72	24.90%
20.		GPLT8			90	23.68%
21		GPLT9			115	11.40%

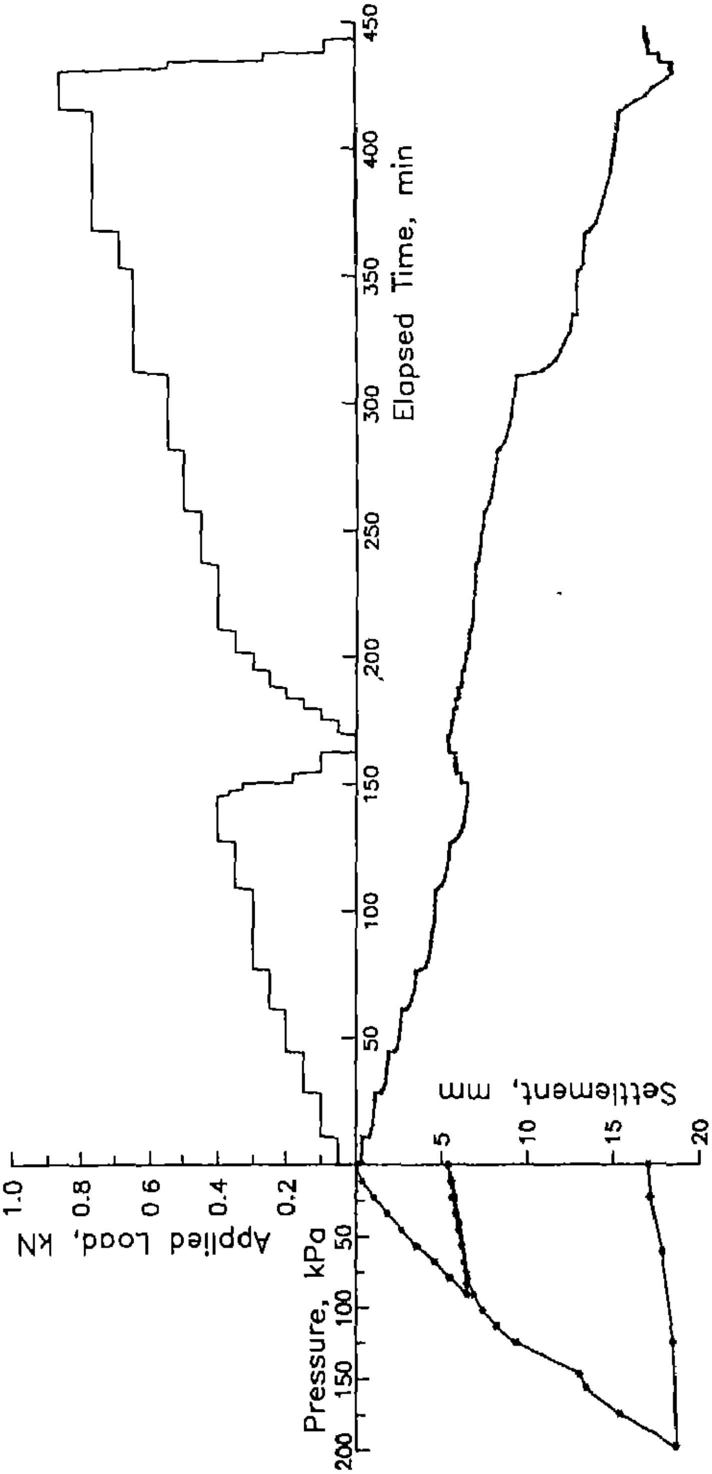

Figure 3.2: *Typical Load Test Results for SPLT5*

3.4.1 Load Tests on no Pile & Single Pile

For load tests on untreated soil, the plate diameter is varied. In the load tests on plate with single pile, the diameters of the plate and of the granular pile are varied. The stress settlement curves obtained from three tests on unreinforced soil and nine load tests with single granular piles for three plate diameters are plotted in Fig 3 3

These plots confirm the fact that the reinforced ground will yield higher ultimate bearing capacity than that for the untreated ground As can be seen from the curves in Fig 3 3, not only the ultimate bearing capacities but also the initial slopes of the stress settlement curves decrease with decrease in area ratio, a_r This may be due to the fact that an increase in the plate diameter results in higher contribution of the surrounding soft soil in the load test

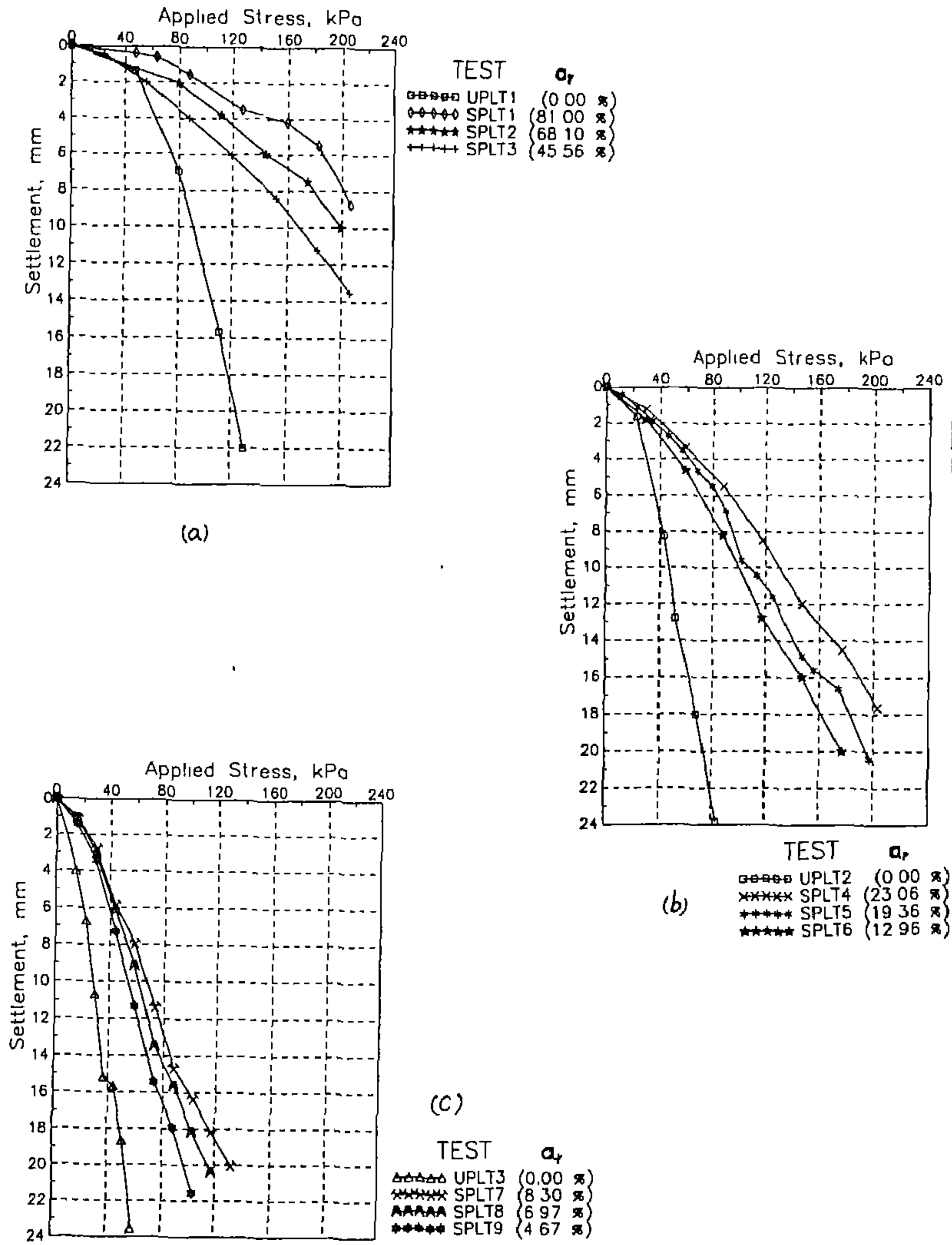

Figure 3.3: *Stress-Settlement curves for Load Tests on Single Pile*

The results of the load tests on plate with single granular piles are also plotted in the form of load-settlement curves in the Fig. 3 4.

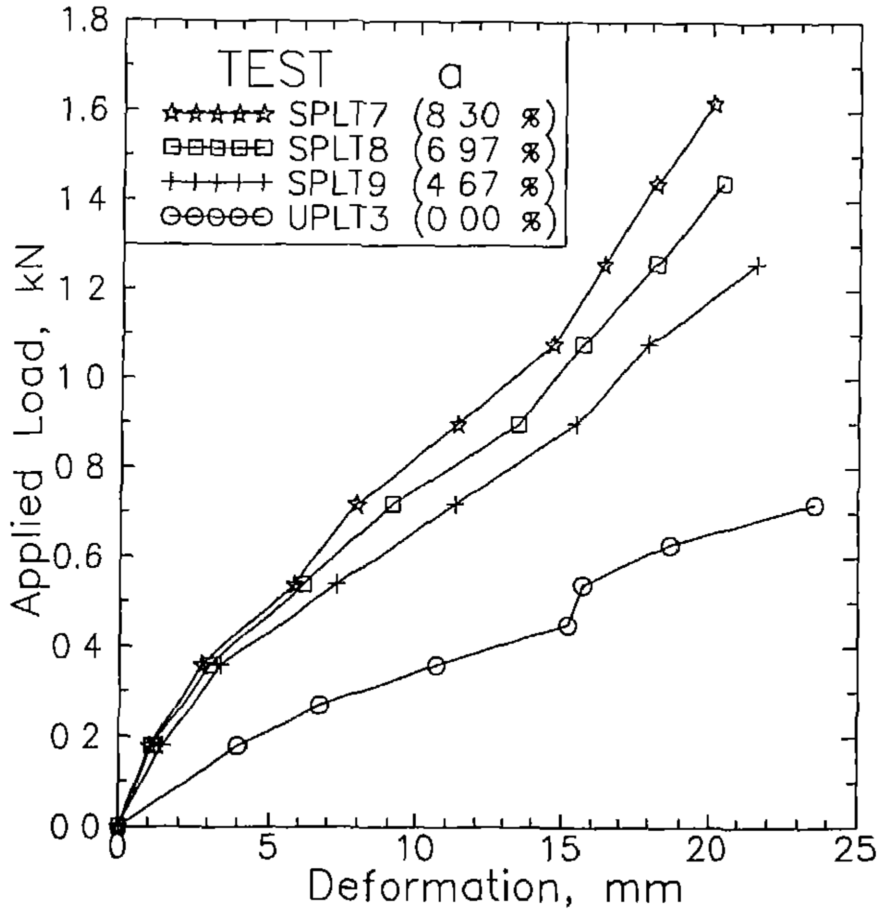

Figure 3.4. *Load-Settlement for different values of Area Ratio*

This plot also shows that with increase in values of area ratio, a_r, the initial slope of the curve and the ultimate load increase for the load tests of the same plate diameter Similar observations can be made for the remaining load tests performed on plates of different diameter.

3.4.2 Load Tests on Pile Groups

All the load tests on plate with group of three granular piles were performed using the plate having diameter of 125 mm The parameters varied are the diameter of granular pile

and the spacing between the piles. The results obtained in the form of stress-settlement curves for three different pile diameters are plotted in Fig 3.5

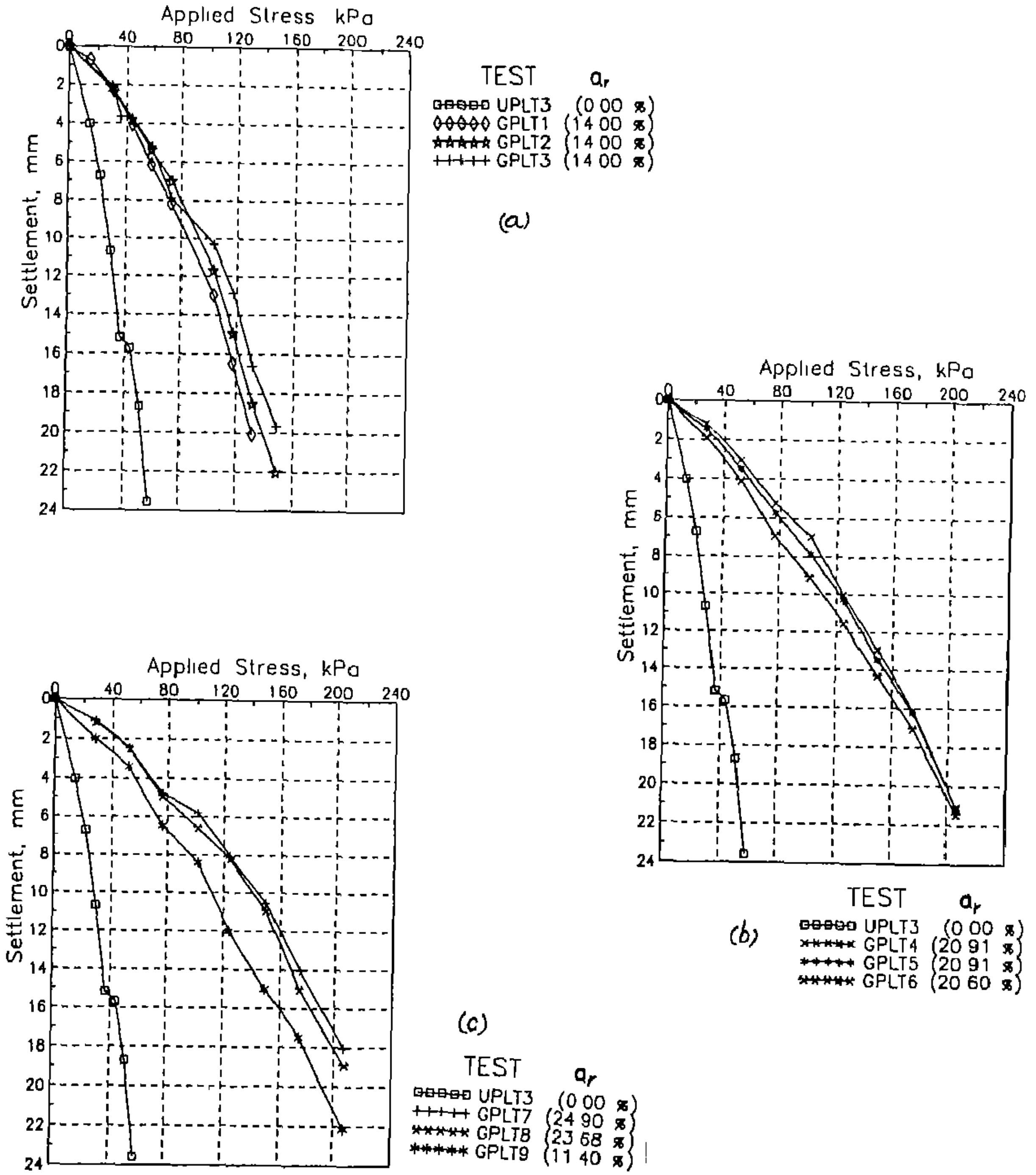

Figure 3.5. *Stress–Settlement curves for Plates with 3 Pile Group*

From these figures it can be seen that nearly all the tests exhibit similar stress-deformation behaviour. Similar to the load test results on plate with single granular pile, with decreasing area ratio, a_r, both the ultimate bearing capacity and initial slopes decrease. There is marked increase in the degree of reinforcement for the case of group of three granular piles

3.5 Interpretation of Load Tests

The stress-displacement curves obtained from all the load tests are plotted in Figs. 3.3 and 3.5. In order to estimate the ultimate bearing capacity and the initial slope, the data was replotted on a hyperbolic plot– δ/q verses δ plot. The initial slopes, k_s and the ultimate bearing capacities, q_{ult} are determined. Further, undrained modulus for untreated, E_u and granular pile treated soil, E_{eq} are determined from stress–immediate settlement curves. All these results are summerized in Table 3.2. Discussions are presented in the following subsections.

Table 3.2: *Summary of Load Tests Results*

Load Test No.	*Type of Test*	*Notation*	*Area Ratio* a_r %	*Subgrade Modulus* k_s, kPa/mm	*Ultimate Bearing Capacity* q_{ult}, kPa	*Undrained Modulus* E_u, MPa
1.		UPLT1	–	8.01	131.02	1.000
2.	Unreinforced	UPLT2	–	3.24	90.82	0.942
3		UPLT3	–	2.49	88.11	1.155
4		SPLT1	81.00%	60.04	581.25	2.38
5		SPLT2	68.06%	41.25	376.00	2.15
6.		SPLT3	45.56%	21.82	505.59	1.82
7.	Single	SPLT4	23.04%	16.42	262.47	1.46
8.	Pile	SPLT5	19.36%	15.23	232.52	1.36
9		SPLT6	12.96%	12.57	200.71	1.27
10.		SPLT7	8.30%	7.59	180.63	1.24
11.		SPLT8	6.97%	6.04	171.82	1.18
12		SPLT9	4.67%	5.45	149.79	1.13
13.		GPLT1	14.00%	10.69	244.95	1.48
14.		GPLT2	14.00%	10.00	248.73	1.36
15		GPLT3	14.00%	9.09	345.04	1.25
16.	Group	GPLT4	20.91%	14.23	437.03	1.76
17.	of	GPLT5	20.91%	12.97	385.04	1.73
18.	Piles	GPLT6	20.60%	11.11	563.91	1.69
19.		GPLT7	24.90%	16.27	451.12	2.05
20.		GPLT8	23.68%	15.00	442.31	1.74
21.		GPLT9	11.40%	10.91	350.67	1.71

3.5.1 Subgrade Modulus

From the results in Table 3.2, it is observed that in addition to area ratio, the subgrade modulus is function of the plate diameter and the number of piles (single pile or group of piles) With decrease in the plate diameter and increase in number of piles, the k_{seq} values are found to increase for the same area ratio, a_r. The ratios of subgrade modulus of reinforced soil to that of untreated soil, k_{seq}/k_s are plotted against area ratio, a_r in the Fig 3 6 This figure shows that with increase in area ratio, a_r, the subgrade modulus ratio, k_{seq}/k_s increases for all the tests There is wide variation in k_{seq} values for the case of load tests with single granular pile which may be due to possible experimental and/or estimation errors. The equations of best fit linear curve for the case of plate load test on

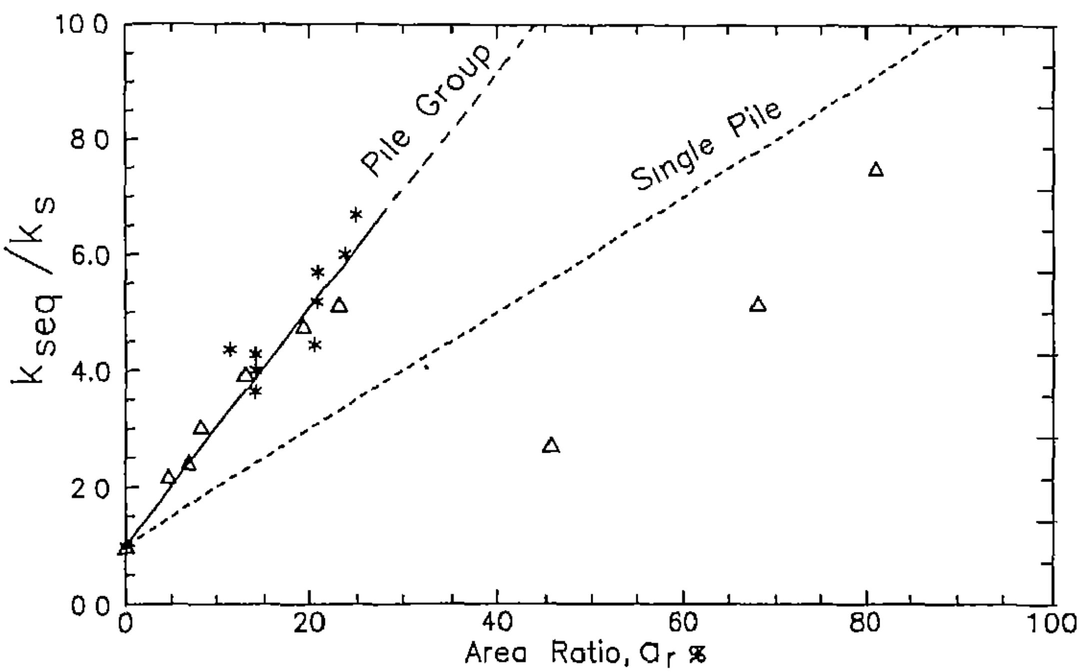

Figure 3.6: *Subgrade Modulus Ratio verses Area Ratio*

plate with single granular pile is

$$k_{seq} = (0.1 \cdot a_r + 1.0) \; k_s \tag{3.1}$$

and that for pile group case is

$$k_{seq} = (0.205 \cdot a_r + 1\,0) \cdot k_s \tag{3.2}$$

where area ratio, a_r is expressed in percentage.

3.5.2 Ultimate Bearing Capacity

The ultimate bearing capacities of treated and untreated soil, q_{ult} are also determined from the hyperbolic plots. For the case of untreated soil, the ultimate bearing capacity values are 131.02, 90.82 and 88.11 kPa corresponding to the plates having diameter 40, 75 and 125 mm, indicating that ultimate bearing capacity depends upon the plate size. This may be due to possible size and side effects during the load tests. With increase in area ratio, ultimate bearing capacity is found to increase, as expected. For the same plate diameter, the bearing capacity ratios, BCR are determined and plotted in Fig. 3.7 against area ratio. With increase in values of area ratio, BCR increases linearly for both

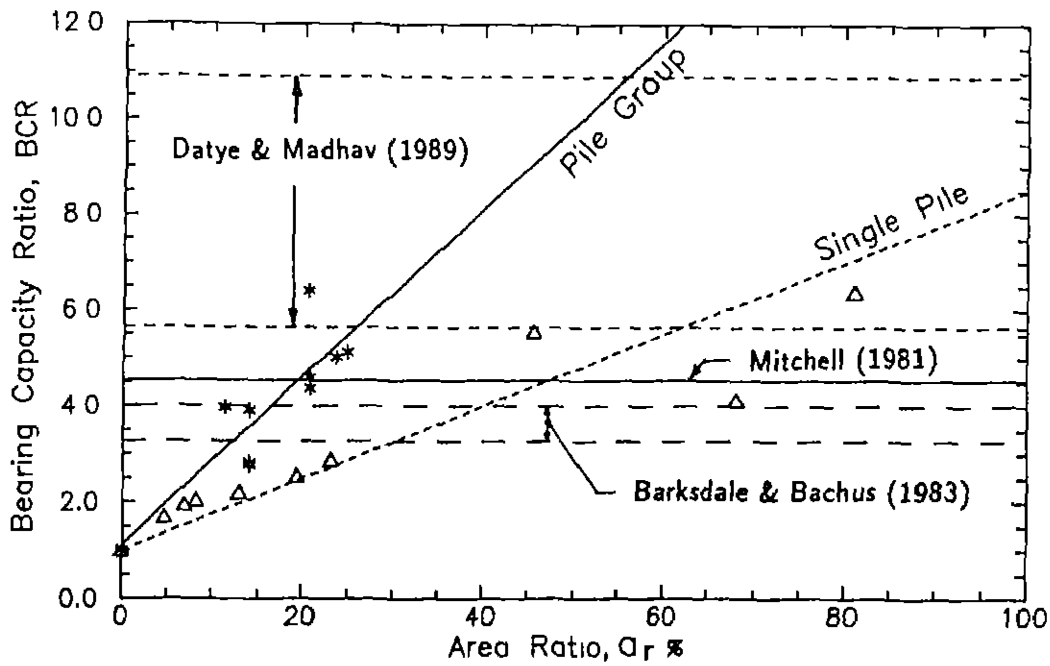

Figure 3.7. *BCR verses Area Ratio*

types of load tests on plate with single granular pile and group of three piles. This plot also shows that for pile group tests, BCR values are greater than those for the single pile tests. The equations for best fit curves for the case of single pile tests is

$$BCR = 0.075 \cdot a_r + 1.0 \tag{3.3}$$

and that for pile groups is

$$BCR = 0.18 \; a_r + 1.0 \tag{3.4}$$

where area ratio, a_r is expressed in percentage.

Bearing capacity ratio obtained from the actual field observations (Datye & Madhav, 1989) and suggested earlier (Mitchell, 1981 and Barksdale & Bachus, 1983) are also plotted. It can be seen that for low values of area ratio, (a_r < 25%), bearing capacity ratios BCR obtained from the tests on plate with single granular pile are less than the ranges of field observed and suggested values. However for higher area ratio, BCR values are in that range. For plate load tests with group of three piles, almost for all values of area ratio, the BCR values are in the range of actually field observed and suggested values.

3.5.3 Undrained Modulus

The total settlement is composed of immediate settlement upon loading and consolidation settlement from the silty soil. For all the tests, the immediate settlement for each loading stage are cumulatively added and results of few tests are plotted against the applied stress as shown in Fig. 3.8. It can be seen these curves are nonlinear, indicating a variation in the

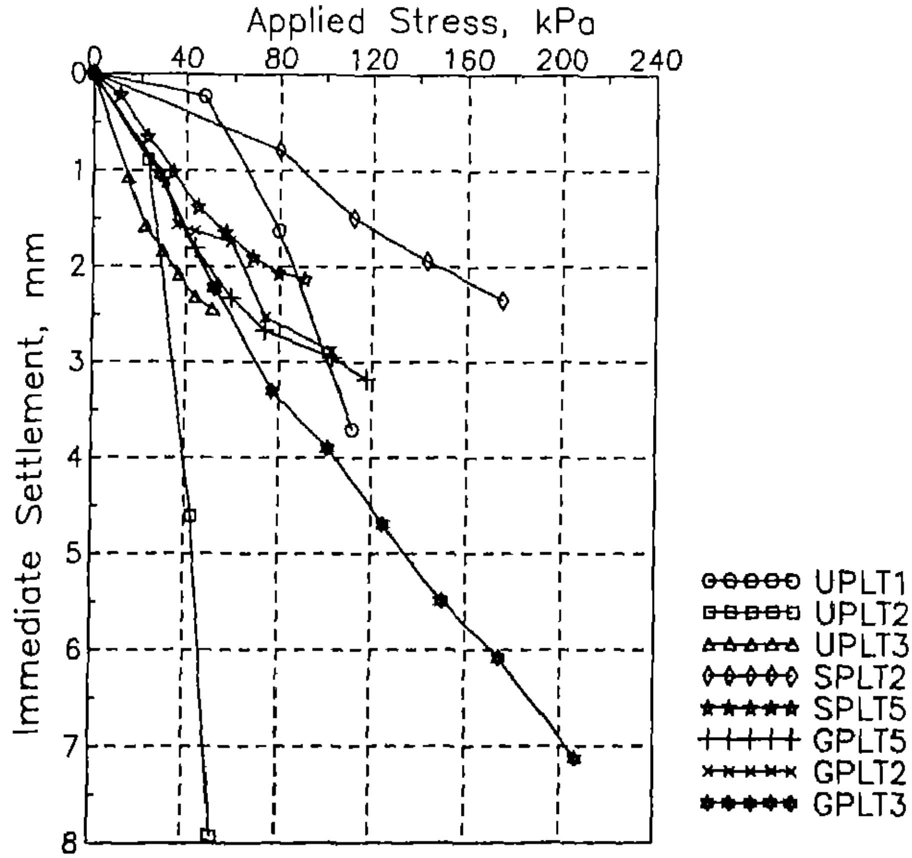

Figure 3.8: *Immediate Settlement verses Applied Stress*

undrained moduli of untreated soil, E_u and equivalent moduli for granular pile reinforced ground, E_{eq} over the considered stress range Knowing that immediate settlement is undrained in nature, the values of undrained moduli - E_u and E_{eq} are calculated using the theory of elasticity for a rigid plate placed at the ground level assuming homogeneous (equivalent) soil conditions. The Poisson's ratio, ν is taken as 0.5. For unreinforced soil E_u has values of 1.00, 0.942 & 1.155 MPa corresponding to plate diameters of 10. 75 and 125 mm, respectively, with an average value of 1.05 MPa. The ratios of deformation moduli of granular pile treated and untreated soils, E_{eq}/E_u for the same plate diameter are plotted against the area ratio, a_r in the Fig 3.9

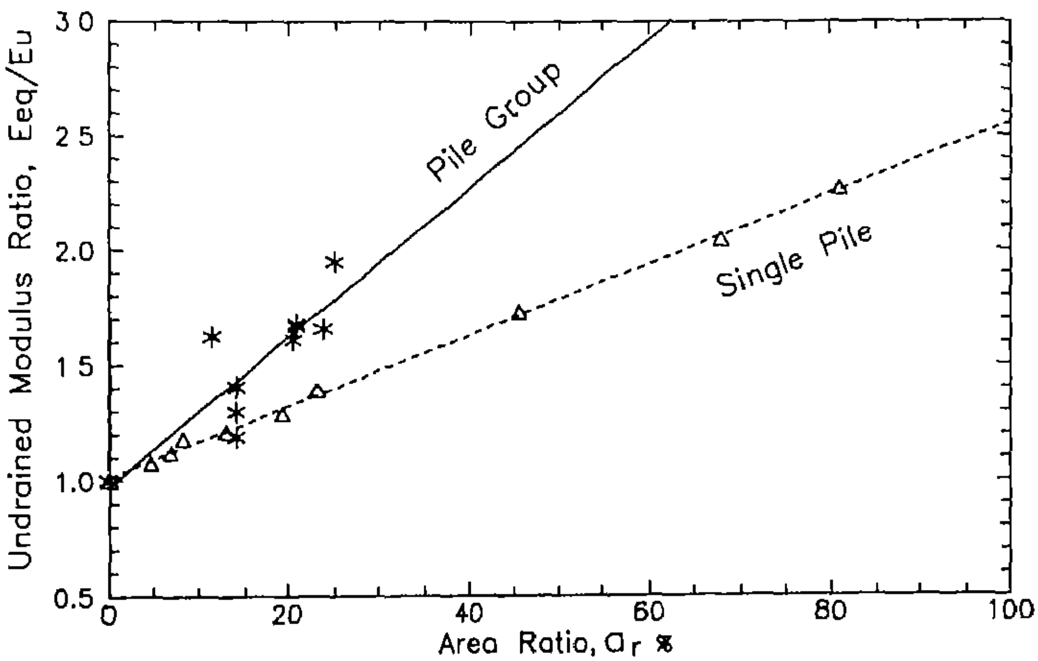

Figure 3.9: E_{eq}/E_u verses *Area Ratio*

It is observed that the modular ratio, E_{eq}/E_u increases linearly with the area ratio, a_r for both the tests with a single pile and with group of three granular piles. For the same area ratio, equivalent undrained modulus obtained from the results of load tests on plate with single pile are observed to be less than those for the case group of three granular piles About 3 times increase in undrained modulus is indicated from pile group tests for $a_r = 63\%$, whereas for the same area ratio, only twofold increase is observed from load tests with single pile. For the case of load tests on plate with single granular pile, the

best fit curve can expressed as

$$E_{eq} = (0.015 \; a_r + 1) \cdot E_u \tag{3.5}$$

and that for group of three piles is

$$E_{eq} = (0.032 \; a_r + 1) \; E_u \tag{3.6}$$

where area ratio, a_r is expressed in the percentage

Based on the analysis of field load tests on sandy soils, Sarma (1994) reports about 6–15 times increase in undrained modulus with area ratio ranging between 15 to 33% This indicates that very small increase in undrained modulus is obtained from small scale model load tests on reconsolidated silty soil deposits

3.6 Shape of Granular Pile after Load Tests

The granular piles were excavated after the load tests to investigate the deformed shape. At each 2 cm of excavation, the diameter of each pile was measured, down to full depth The typical deformed shape is plotted in Fig 3.10.

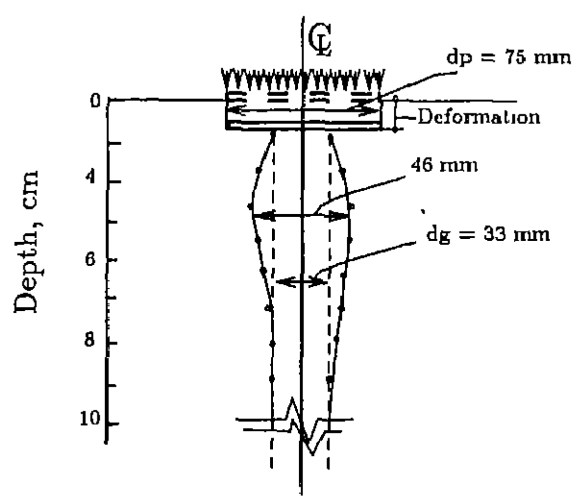

Figure 3.10: *Typical Shape of Granular Pile after Load Test*

It was observed that the maximum bulge occurred near the top of pile and ranged from 16 to 58 mm below the top surface. Comparing with the initial pile diameter of 33 mm, the measurements are in close agreement with the observations of Hughes *et al* (1975) and Bergado & Lam (1987) that the maximum bulge occurred near the top surface at depth of approximately equal to one-half to one pile diameter.

3.7 Conclusions

Based on the results of reconsolidation of the soil and model load tests, following conclusions can be made

- For model load tests, *reconsolidation technique* offers the best choice to prepare small scale, uniform and identical soil deposits.
- The total settlement during the reconsolidation process are about 20-22% of the total height of the slurry sample with initial water content equal to 1.2 times the liquid limit.
- The required consolidation time is predictable.
- All the total stress applied during reconsolidation process was not the virgin stress.
- Based on results of water content profile, high uniformity of laboratory prepared soil beds was achieved.
- Due to dry installation method of granular piles, surface heave was observed and the actual diameter of installed granular pile was greater than the pipe diameter.
- For all the load tests, with increase in area ratio, the subgrade modulus, ultimate bearing capacity and undrained modulus are observed to increase.
- For same area ratio, the values of subgrade modulus, ultimate bearing capacity and undrained modulus from the load tests on plate with single granular pile are greater than those for the load tests with group of three granular piles.
- Subgrade modulus of the granular pile reinforced soil is observed to decrease with increase in plate diameter.

- Bearing capacity ratio from load tests on plate with single granular pile is less than the field observed and suggested values for low area ratio. However from the results of load tests with group of three piles, the BCR values are in the range of actually observed and suggested values

- Very small increase in undrained modulus is observed for all the load tests

- The observed bulging of granular piles near the top surface at depth of approximately equal to one to'one-half of pile diameter is similar to the reported observations.

Chapter 4

BEARING CAPACITY

4.1 General

As a ground improvement method, granular pile reinforcement technique has five purposes. to enhance bearing capacity, to reduce deformations, to accelerate the rate of primary consolidation, to increase shearing resistance and to reduce susceptibility to liquefaction. The reduction of settlements rather than the bearing capacity of granular piles reinforced ground is generally the primary consideration while designing the structures. However there may be situations where bearing capacity calculations are critical such as for embankments, heavy tanks, or similar structures. Hence the determination of bearing capacity of a soil reinforced by a group of granular piles is an important consideration in the design process. In the present Chapter, an attempt is made to determine the bearing capacity of such a composite ground assuming that loading is through a rigid footing and the failure surface is circular.

4.2 Bearing Capacity of Composite Ground

From literature review, it is found that the design calculations concerning the bearing capacity of such reinforced soils have been mainly presented for single, isolated granular pile based on different modes of failure. Bergado *et al.* (1991) tabulate the different approaches suggested by various researchers to determine the ultimate bearing capacity of such a composite ground. Design calculations concerning such reinforced soils have been presented assuming trench like reinforcement under plane strain assumption (Madhav &

Vitkar, 1978) and for single isolated granular pile assuming axisymetric condition (Hughes & Withers, 1975)

Very few attempts have been made for the case of a soil reinforced by a group of granular piles For stiff and very stiff cohesive soils, Barksdale & Bachus (1983) reported an approach based on an assumption that the soil immediately beneath the foundation fails along a planer rupture surface, forming a triangular block To evaluate the shear resistance developed along the failure surface, they presented a procedure to determine average shear parameters of the composite soil. Further they suggested a method applicable for a group of granular piles in soft soils similar to single, isolated granular pile In this method, ultimate bearing capacity of composite soil is given by

$$q_{ult} = N_c \cdot c_u \tag{4.1}$$

where, N_c is a bearing capacity factor and c_u is undrained shear strength of surrounding soil Barksdale & Bachus (1983) suggested the range of factor N_c to be between 18 and 22. depending upon the compressibility of in-situ soil Earlier, Mitchell (1981) recommended a value of N_c of 25 for vibro-replacement stone columns Based on number of case histories, Datye & Madhav (1989) found that the factor N_c ranges between 31 and 60 for the rammed stone columns. Of course, the equipment, construction technique and type of in-situ soil have a significant influence on the factor N_c

Enoki *et al* (1991) presented a method to determine equivalent shear parameters of composite soil and carried out bearing capacity analysis assuming infinite extent of the reinforced ground beneath a rigid footing. However their analysis using equivalent soil parameters overestimates bearing capacity for the low values of area ratio and underestimates for higher values of area ratio when compared with conventional approach of equivalent soil parameters and the method suggested by Aboshi *et al* (1979).

Bouassida *et al.* (1995) presented an approximate lower bound solution of the bearing capacity of composite soil mass using the equivalent soil parameters similar to those reported earlier by Enoki *et al* (1991). As expected, the reported equivalent cohesion reduces with increasing values of area ratio and frictional angle of column material (ϕ_g). However, for values of $\phi_g > 30^o$, the equivalent cohesion is found to increase rather than decrease Also, the expression given for the determination of equivalent cohesion does not satisfy the end condition for the case of fully reinforced ground i. e. for $a_r = 1.0$ This may be due to the assumption made regarding the determination of lateral confining

pressure on the granular piles. Further their analysis is observed to be independent of the geometry (width) of the footing.

These studies indicate that there is need to develope an approach which can predict the appropriate improvement in the bearing capacity of composite soil and takes into account the effect of failure surface and of the geometry of the footing In the present analysis, bearing capacity problem of a soil reinforced with a group of granular piles is converted to that of an equivalent soil problem and is solved by using two different approaches

4.3 Problem Formulation

The bearing capacity of a soft soil of depth, D, reinforced by the granular piles over a width, B (Fig 4.1) is considered.

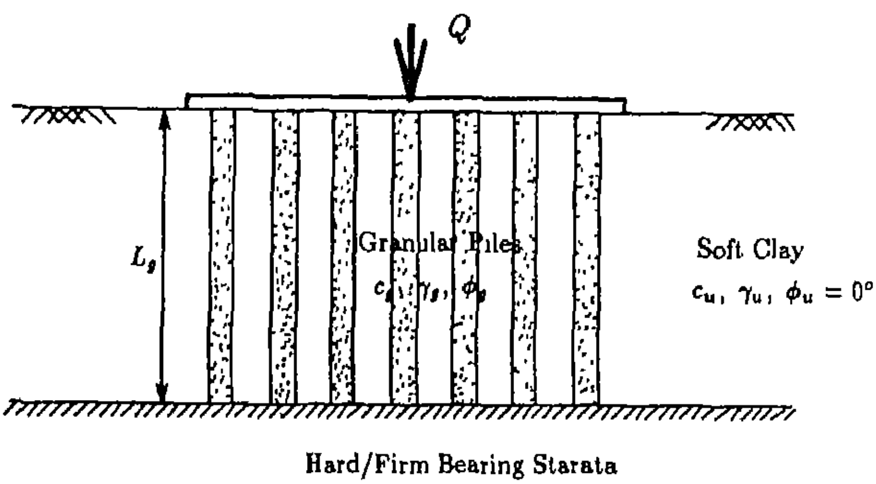

Figure 4 1: *Bearing Capacity of Reinforced Ground*

In order to achieve a result that could be applied to a wide range of practical situations, the approach is not restricted to a particular shape of the footing or to a particular distribution of the granular piles. However for the sake of simplicity, all the granular piles are supposed to have the same diameter and spacing and installed in the same pattern (square or triangular). The granular piles have length L_g, diameter d_g, and spacing s Depending upon the pattern of piles, area ratio can be determined as

$$a_r = c_s \, (d_g \, / \, s)^2$$

where, c_s = 1.270 for triangular; and

= 1.013 for square patterns.

If the foundation is loaded rapidly, the undrained shear strength, c_u is developed in the cohesive soil while the cohesion of the granular pile material is zero. The reinforcement effect is from the frictional resistance of the granular pile material.

To determine the bearing capacity of a structure resting on such a composite soil, the reinforced soil is replaced by an equivalent soil as shown in Fig 4.2 of the same extent

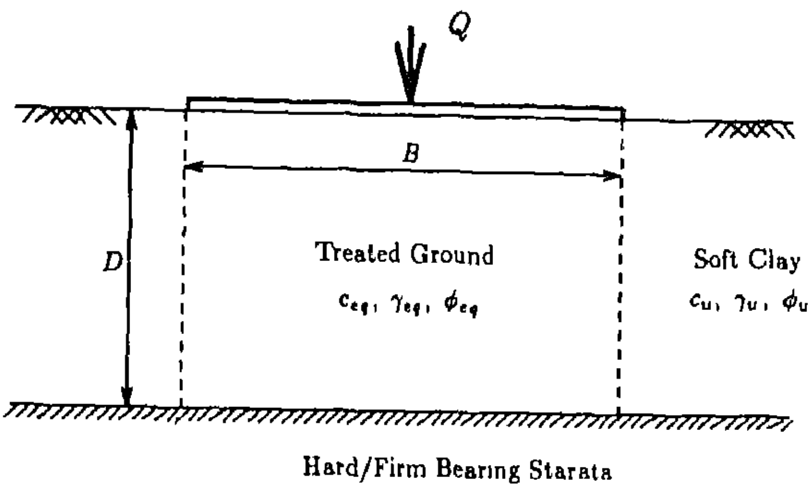

Figure 4.2: *Bearing Capacity of an Equivalent Soil*

whose shear strength parameters are determined as explained in the following Section

4.3.1 Determination of Equivalent Soil Parameters

In the present analysis, the shear strength of improved ground is determined based on the concept of equivalent anisotropic $c - \phi$ ground as given by Enoki *et al.* (1991) The procedure to obtain c_{eq}, ϕ_{eq} and γ_{eq} is described as below.

Fig. 4.3 shows a unit cell of improved ground and is composed of a single granular pile and the in-situ soil. The vertical stress σ_v is the the weighted sum of stress σ_{vg} on the granular pile and σ_{vc} on the surrounding clay with respect to their areas as shown in Fig. 4.3 and expressed as

$$\sigma_v = a_r \cdot \sigma_{vg} + (1 - a_r) \cdot \sigma_{vc} \quad (4.2)$$

where, a_r denotes area ratio which represents the degree of improvement of the foundation bed Assuming individual failures of pile material and surrounding clay, the Mohr-Coloumb's criterion gives

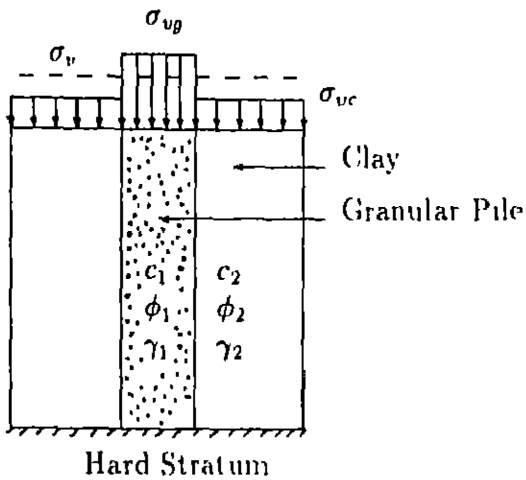

Figure 4.3 *Unit Cell*

$$\sigma_{vg} = \sigma_h N_1 + 2 c_1 \sqrt{N_1} \tag{4.3}$$

$$\sigma_{vc} = \sigma_h N_2 + 2 c_2 \sqrt{N_2} \tag{4.4}$$

where $N_1 = \tan^2(45^o + \phi_1/2)$

and $N_2 = \tan^2(45^o + \phi_2/2)$

In above expressions c_1, c_2 and ϕ_1, ϕ_2 denote cohesions and angles of shearing resistance of granular pile material and clay respectively.

From equations 4.2, 4.3 and 4.4 one gets

$$\begin{aligned}\sigma_v = {} & \sigma_h [(N_1 - N_2) a_r + N_2] \\ & + 2 [c_1 a_r \sqrt{N_1} + c_2 (1 - a_r) \sqrt{N_2}]\end{aligned} \tag{4.5}$$

For improved ground with equivalent cohesion, c_{eq} and equivalent frictional angle, ϕ_{eq}, the failure criterion becomes

$$\sigma_{vc} = \sigma_h N_{eq} + 2 c_{eq} \sqrt{N_{eq}} \tag{4.6}$$

Comparing equations 4.5 and 4.6 one gets

$$N_{eq} = a_r N_1 + (1 - a_r) N_2 \tag{4.7}$$

and

$$c_{eq} = \frac{c_1\, a_r\, \sqrt{N_1} + c_2\, (1 - a_r)\sqrt{N_2}}{\sqrt{a_r\, N_1 + (1 - a_r)\, N_2}} \tag{4.8}$$

Assuming that the granular pile material has no cohesion ($c_1 = c_g = 0$) and that the soft soil is in an undrained state ($\phi_2 = \phi_u = 0$), the above equations reduce to

$$c_{eq} = (1 - a_r)\, c_u \sqrt{K} \tag{4.9}$$

$$\phi_{eq} = \pi/2 - 2\tan^{-1}\sqrt{K} \tag{4.10}$$

where, the constant K is

$$\begin{aligned} K &= 1/\sqrt{N_{eq}} \\ &= \frac{1 - \sin\phi_g}{1 + (2\,a_r - 1)\sin\phi_g} \end{aligned} \tag{4.11}$$

Equivalent unit weight within the composite ground is given by

$$\gamma_{eq} = a_r\, \gamma_u + (1 - a_r)\, \gamma_g \tag{4.12}$$

The treated ground is replaced by an equivalent soil with above parameters but of the same extent.

4.3.2 Determination of Bearing Capacity

In the present analysis, the problem of undrained bearing capacity of a reinforced soil with a group of granular piles is reduced to that of an equivalent problem as shown in Fig 4.2 and its solution is obtained by considering the two approaches as described below·

4.3.3 Approximate Lower Bound Solution

In this simple, static approach, no particular slip surface is considered. At the limiting condition, the principal stresses for the elements in the equivalent soil zone can be represented by the stresses acting on the element **1** as shown in Fig 4.4. In the outside zone, the major principal stress on the element **2** is horizontal. When the two elements are adjacent to each other at the vertical interface, it is evident that $\sigma_{1,3} = \sigma_{1,2}$, with a principle stress rotation of 90° between the two elements.

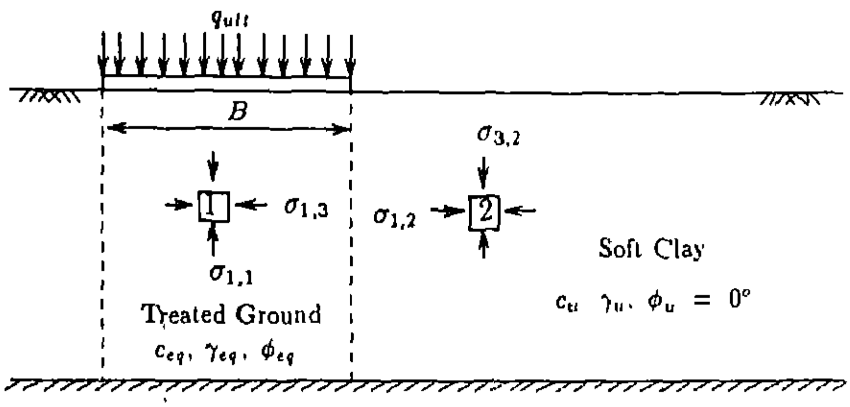

Figure 4.4. *Approximate Lower Bound Solution*

For element **2**, $\phi_2 = \phi_u = 0°$. Hence one gets the major principle stress as

$$\sigma_{1,2} = 2 \cdot c_u = \sigma_{3,1} \tag{4.13}$$

For element **1**, just under the footing, the major principle stress $\sigma_{1,1}$ is

$$\sigma_{1,1} = q_{ult} = \sigma_{3,1} \cdot \tan^2(45 + \phi_{eq}/2) + 2\, c_{eq} \cdot \tan(45 + \phi_{eq}/2) \tag{4.14}$$

which reduces to

$$q_{ult} = 2\, c_u \cdot \tan^2(45 + \phi_{eq}/2) + 2\, c_{eq} \cdot \tan(45 + \phi_{eq}/2) \tag{4.15}$$

Further simplifying,

$$\frac{q_{ult}}{c_u} = 2\left[N_{eq} + \frac{c_{eq}}{c_u}\sqrt{N_{eq}}\right] \tag{4.16}$$

$$\frac{q_{ult}}{c_u} = 2\left[\frac{1 + (1 - a_r) \cdot K\sqrt{K}}{K^2}\right] \tag{4.17}$$

where, the constant K is

$$\begin{aligned} K &= 1/\sqrt{N_{eq}} \\ &= \frac{1 - \sin\phi_g}{1 + (2\,a_r - 1)\sin\phi_g} \end{aligned} \tag{4.18}$$

For no granular pile reinforcement, $a_r = 0.0$, and $\phi_{eq} = \phi_u = 0° \Rightarrow K = 1.0$. Hence the ultimate bearing capacity for untreated ground reduces to

$$\frac{q_{ult,ur}}{c_u} = 2\,[1 + 1] = 4 \tag{4.19}$$

Hence the bearing capacity ratio, BCR is expressed as

$$BCR = \frac{q_{ult}}{q_{ult,ur}} = \frac{1}{2}\left[\frac{1 + (1 - a_r) \;\; K\sqrt{K}}{K^2}\right] \tag{4.20}$$

From the above expression, it can be seen that the increase in bearing capacity is a function of area replacement ratio, a_r and the angle of shearing resistance of granular pile material, ϕ_g

4.3.4 Limit Equilibrium Approach

In this method it is assumed that when the footing is loaded, to produce the maximum bearing pressure, q_{ult}, it will rotate about some center of rotation with the shear resistance being developed along the perimeter of a circular surface (Fig 4 5)

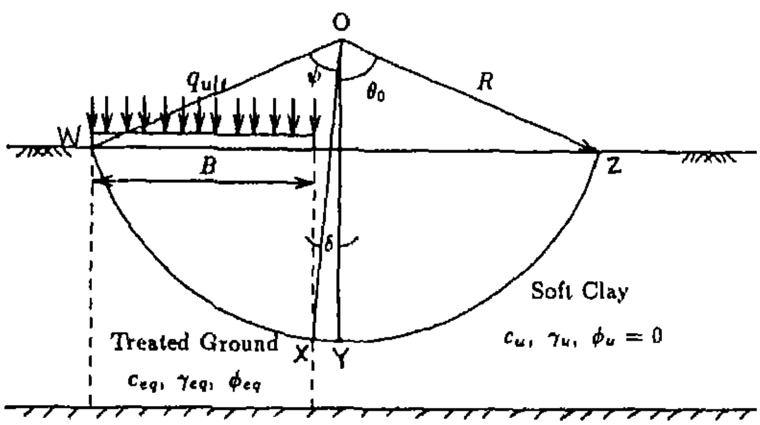

Figure 4.5: *Limit Equilibrium Approach*

The minimum value of q_{ult} is obtained by making search for the critical slip surface. The slip surface is assumed to be circular and starts from one of the edges of the footing as shown in Fig. 4.9, where R and θ_0 are the radius and half the angle subtended at the center of the trial arc. Taking moments of all forces about the center, O, and summing for equilibrium;

$$q_{ult}\,(R\,sin\theta_0 \;-\; B/2)\,B = \int_0^{\theta_0+\delta} R^2\,c_u \cdot d\theta \;+\; \int_0^{\psi} R^2\,c_{eq} \cdot d\theta \;+\; M_{WX} \tag{4 21}$$

where,

M_{WX} = Moment of frictional comp. of shear resistance along arc $\widehat{WX}$

$$= \text{Moment along arc } \widehat{W'Y} - \text{Moment along arc } \widehat{XY}$$

$$= R \cdot \int_0^{\theta_0} (R d\theta \cdot \cos\theta) \; R(\cos\theta \; - \; cos\theta_0) \; (\gamma_{eq} \;\; cos\theta) \; \tan\phi_{eq}$$

$$(R\sin\theta_0 \; - \; B) \cdot (R \; - \; R\cos\theta_0) \;\; \gamma_{eq} \;\; \tan\phi_{eq} \cdot R$$

$$= R^3 \gamma_{eq} \tan\phi_{eq} \left[\frac{B}{R}(1 - \cos\theta_0) - \frac{\sin^3\theta_0}{3} - \cos\theta_0 \left(\frac{\theta_0}{2} + \frac{\sin 2\theta_0}{4} - \sin\theta_0 \right) \right]$$

Hence equation 4 21 becomes as,

$$\frac{q_{ult}}{c_u} \left(\frac{R}{B} \sin\theta_0 - \frac{1}{2} \right) \;=\; (R/B)^2 \left[(\theta_0 + \delta) \; + \; (\theta_0 - \delta) \frac{c_{eq}}{c_u} \right] \; + \; (R/B)^3 \;\; \frac{\gamma_{eq} \; B}{c_u} \tan\phi_{eq}$$
$$\times \left[\frac{B}{R}(1 - \cos\theta_0) - \frac{\sin^3\theta_0}{3} - \cos\theta_0 \left(\frac{\theta_0}{2} + \frac{\sin 2\theta_0}{4} - \sin\theta_0 \right) \right] \quad (4.22)$$

which can be written as,

$$q_{ult} \;=\; N_c^* \;\; c_u \quad (4\ 23)$$

where,

$$N_c^* \;=\; (\overline{R})^2 \cdot \left[(\theta_0 + \delta) \; + \; (\theta_0 - \delta) \, \overline{C} \right] \; + \; (\overline{R})^3 \cdot N_\gamma^* \cdot \tan\phi_{eq} \cdot$$
$$\times \left[(1/\overline{R})(1 - \cos\theta_0) - \frac{\sin^3\theta_0}{3} - \cos\theta_0 \left(\frac{\theta_0}{2} + \frac{\sin 2\theta_0}{4} - \sin\theta_0 \right) \right]$$

with $\overline{R} \;=\; R/B, \overline{C} \;=\; c_{eq}/c_u$ and $N_\gamma^* \;=\; \gamma_{eq} \cdot B/c_u$

The expression for N_c^* is minimized with respect to the radius of the arc of the slip surface, R, and half the angle, θ_0, subtended at the center of the trial arc A one point iterative search method is used to determine the parameters - critical radius, R_{cr} and critical angle, θ_{cr}, corresponding to the minimum bearing capacity of such a reinforced soil. The ultimate bearing capacity of the footing lying on such a soil, q_{ult} is determined from the equation 4.22 Taking the bearing capacity of the unreinforced ground as 5.14 c_u, the bearing capacity ratio, BCR is expressed as,

$$BCR \;=\; \frac{q_{ult}}{q_{ult,ur}} \;=\; \frac{q_{ult}}{5.14 \; (c_u)} \quad (4.24)$$

4.4 Results

In this section, the comparsion is made for different approaches for determination of equivalent soil parameters. Charts are presented for determination of these equivalent soil parameters. Further the present approaches for the determination of the bearing capacity of granular pile reinforced soil are compared.

4.4.1 Comparison of Equivalent Parameters' Approach

As mentioned earlier, Priebe (1976) has suggested an approach to determine equivalent composite strength parameters of the treated ground which requires the area ratio as well as the diameter of granular pile to be provided. Further Barksdale & Bachus (1983) report a conventional method for determination of equivalent soil parameters based on area ratio only in which

$$c_{eq} = (1 - a_r)\, c_u \tag{4.25}$$

$$\phi_{eq} = \tan^{-1}(a_r \, \tan\phi_g) \tag{4.26}$$

assuming that the $\phi_u = 0^o$ & $c_g = 0$ kPa. To take into account the effect of ϕ_u on ϕ_{eq}; equation (4.13) can be modified as

$$\phi_{eq} = \tan^{-1}[a_r \, \tan\phi_g + (1 - a_r)\tan\phi_u] \tag{4.27}$$

Using the approach given by Enoki *et al.* (1991), for selected values of soil parameters of in-situ soil and the granular pile and area ratio, the equivalent soil parameters are determined and compared to the same obtained by using other two approaches as shown in the Table 4.1. From the table it can be seen that the equivalent soil parameters obtained from three different approaches do not vary significantly. However for $\phi_u = 0°$, Priebe's (1976) approach predict very low equivalent cohesion and very high equivalent frictional angle. For values of $\phi_u > 0^o$ this approach predicts the highest ϕ_{eq} and intermediate c_{eq}. Hence Priebe's (1976) approach is applicable only to the soils having $\phi_u > 0^o$. The modified conventional method gives the highest c_{eq} and lowest ϕ_{eq} for all values of ϕ_u considered. The equivalent soil parameters obtained from the approach suggested by Enoki *et al.* (1991) range between those obtained from the other two methods. Hence this approach appears to be more appropriate to all types of soils.

Table 4.1: *Comparison of Equivalent Soil Parameters*

Soil Parameters	Area Ratio a_r (%)	Equiv. Soil c_{eq} (kPa)	Equiv. Soil ϕ_{eq} (°)	Approach	Reference
$c_u = 4\ kPa$		2.40	22.40	Priebe (1976)	Priebe
$\phi_u = 8^o$, $\phi_g = 40^o$	17.14	3.31	14.59	modified conventional	(1976)
$c_g = 0\ kPa$		2.78	17.86	Enoki *et al.* (1991)	[undrained]
$c_u^{'} = 0\ kPa$		0.0	26.80	Priebe (1976)	Almeida *et al*
$\phi_u^{'} = 23^o$, $\phi_g^{'} = 45^o$	4.90	0.0	24.36	modified conventional	(1985)
$c_g^{'} = 0\ kPa$		0.0	24.92	Enoki *et al.* (1991)	[drained]
$c_u = 15\ kPa$		4.81	25.46	Priebe (1976)	Present
$\phi_u = 0^o$; $\phi_g = 35^o$	30.00	10.50	11.86	modified conventional	Analysis
$c_g = 0\ kPa$		9.68	12.26	Enoki *et al* (1991)	[undrained]

4.4.2 Equivalent Soil Parameters

The variation of the ratio of equivalent cohesion to undrained cohesion of soft soil, c_{eq}/c_u, with area replacement ratio, a_r is plotted in Fig 4.6. The equivalent cohesion, c_{eq} turns

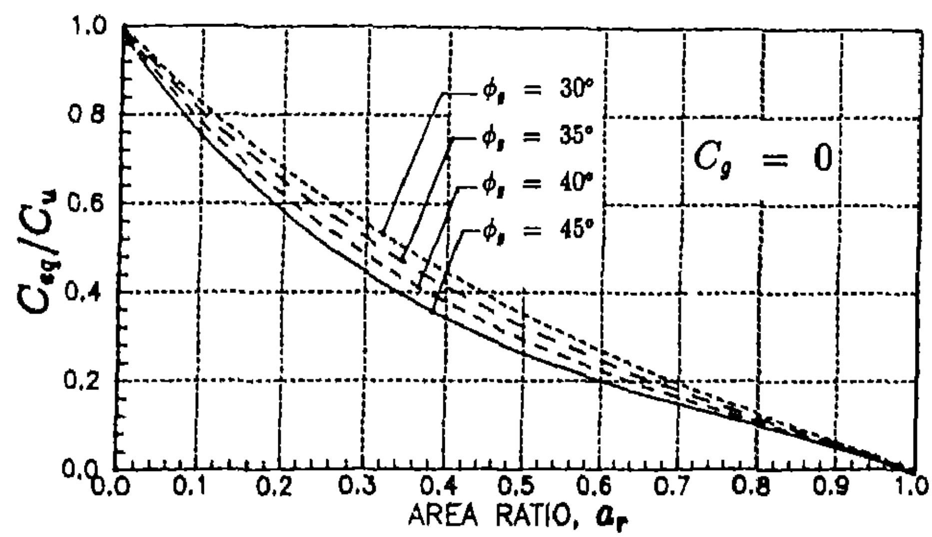

Figure 4.6: *Equivalent Cohesion c_{eq}*

out to be smaller than the undrained cohesion of the untreated soil for all values of the area replacement ratio. The reinforcement effect is entirely due to the frictional properties of the granular pile material and can be estimated by the equivalent friction angle, ϕ_{eq}.

Fig 4.7 presents the variation of equivalent angle of shearing resistance, ϕ_{eq} with area

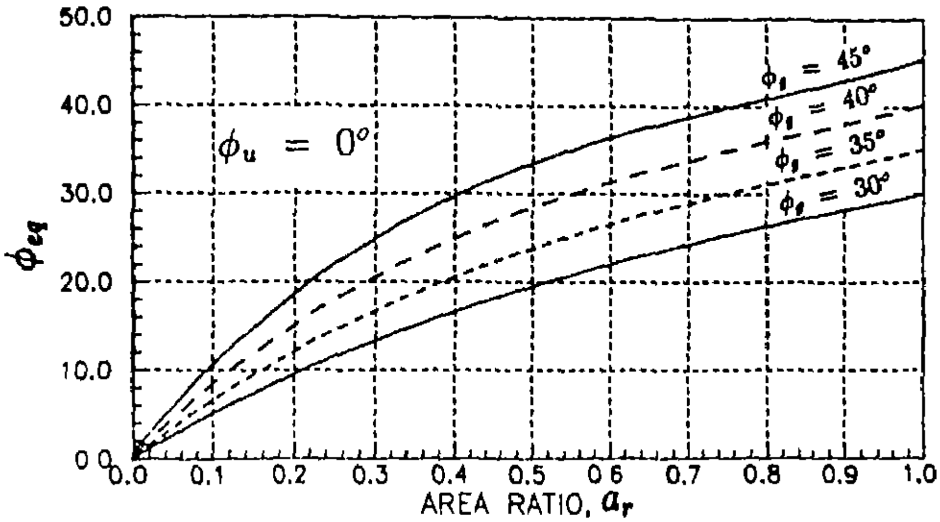

Figure 4.7. *Equivalent Angle of Shearing Resistance*, ϕ_{eq}

ratio, a_r for different values of ϕ_g. As expected, the equivalent angle of shearing resistance increases with increasing values of both a_r and ϕ_g.

The charts shown in Fig 4.6 and Fig 4.7 can be used for the rapid determination of equivalent shear parameters of the composite soil From these charts, for area ratio, $a_r = 0$, the equivalent shear parameters are - $c_{eq} = c_u$ and $\phi_{eq} = \phi_u = 0°$ which are same as that for the cohesive soil (untreated ground). For $a_r = 1.0$, $c_{eq} = 0$ and $\phi_{eq} = \phi_g$, reducing to that for the granular pile material. Hence, for both the equivalent shear parameters c_{eq} & ϕ_{eq}, the approach suggested by Enoki *et al* (1991), satisfies the end conditions.

However, the concept of equivalent soil should be handled very cautiously (Bouassida *et al.*, 1995). It relies on the assumption that the reinforcing inclusions are regularly distributed throughout the soil mass, and that the spacing between two successive inclusions can be considered small enough when compared to the characterstic length of the problem, such as the width of the footing or the width of the loaded area.

4.4.3 Comparison of the Bearing Capacity Approaches

In this Chapter, two different approaches for the determination of increase in the bearing capacity of reinforced ground with granular piles are presented. The approximate lower

bound method predicts increase in bearing capacity which is a function of the area replacement ratio, a_r and the angle of shearing resistance of the granular pile material, ϕ_g. This approach is found to be independent of the geometry of the footing. In the limit equilibrium approach, a search is made for the critical radius and the critical angle subtended at the center of the rotation, to obtain the least value of the bearing capacity of the improved ground. The bearing capacity of the reinforced ground from limit equilibrium approach is observed to be function of the width of the footing also.

A problem involving the determination of bearing capacity of a soil reinforced with a group of granular piles is considered, and the results obtained by using different methods are compared with the present approaches as shown in Fig. 4.8.

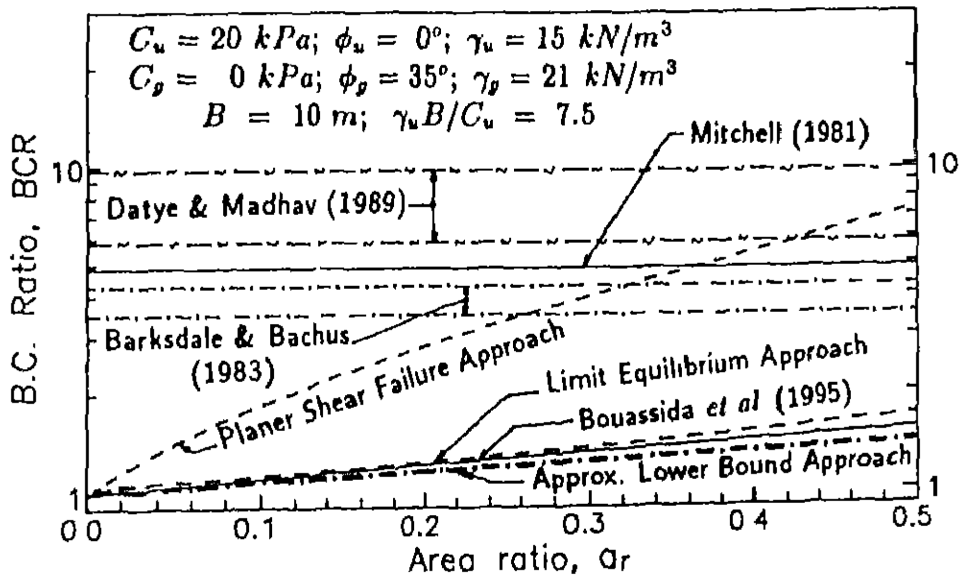

Figure 4.8: *Comparison of Different Approaches*

The approximate lower bound solution gives the least increase in the bearing capacity ratio for the case considered. The approach given by Bouassida *et al.* (1995) predicts an increase in bearing capacity of the composite soil somewhat higher than the approximate lower bound approach. This seems to be due to their assumption made while determining the lateral pressure acting on the granular pile. The increase in the bearing capacity of the reinforced ground obtained by using the limiting equilibrium approach is observed to be higher than the other two approaches. The planer shear failure approach given by Barksdale & Bachus (1983) gives higher values of bearing capacity ratio than all of these three approaches for all values of area ratio considered.

Bearing capacity ratio obtained from the actual field observations (Datye & Madhav, 1989) and suggested earlier (Mitchell, 1981 and Barksdale & Bachus, 1983) are also plotted. It can be seen that all of these theoretical approaches underestimate the values of bearing capacity ratio, BCR.

This may be due to the fact that due to installation of granular piles, the strength properties of the in-situ soil get modified Also it is possible due to consideration of a lower value of the angle of shearing resistance of the granular pile material than the actual field value. The exact determination of the actual in-situ diameter of the granular pile is also difficult which may lead to underestimation of the area replacement ratio All of these aforementioned factors have major influence on the shearing resistance of the composite ground However these approaches may be useful to study the effect of other variables on the bearing capacity improvement.

4.5 Parametric Study

In order to study the effect of various parameters on the bearing capacity of a soil reinforced with a group of granular piles, a parametric study is performed using limit equilibrium approach as it takes into account the effect of the width of the footing In this analysis it is assumed that, the extent of the treated ground is the same as the base width of the footing and the depth of the soft soil layer is greater than the possible maximum depth of the critical circular slip surface. The various parameters considered are

for Footing.

- Base Width, $B = 2$ to 30 m.

for Soft Soil:

- Unit Weight, $\gamma_u = 15$ kN/m^3;
- Undrained Cohesion, $c_u = 20, 30, 40$ kPa;
- Angle of Shearing Resistance, $\phi_u = 0^o$.

for Granular Pile:

- Unit Weight, $\gamma_g = 21$ kN/m^3;
- Cohesion of Granular Pile material, $c_g = 0$ kPa;

- Angle of Shearing Resistance, ϕ_g = 35°, 40° & 45°,
- Area Ratio, a_r = 0.0, 0.1, 0.2, 0.3, 0.4 & 0.5

The variations of the critical radius R_{cr}/B and the critical angle, θ_{cr} (half the angle subtended at the center of the rotation), with the other parameters such as – area replacement ratio, a_r, cohesion ratio, c_{eq}/c_u, equivalent angle of shearing resistance, ϕ_{eq}, the non-dimensional factor, γ_{eq} B/c_u, *etc.* are studied. Further the effect of these parameters on the improvement in bearing capacity is also studied.

4.5.1 Effect of Area Ratio on R_{cr} & θ_{cr}

In Fig. 4.9 variation of the critical radius R_{cr} and the critical angle θ_{cr} are plotted for

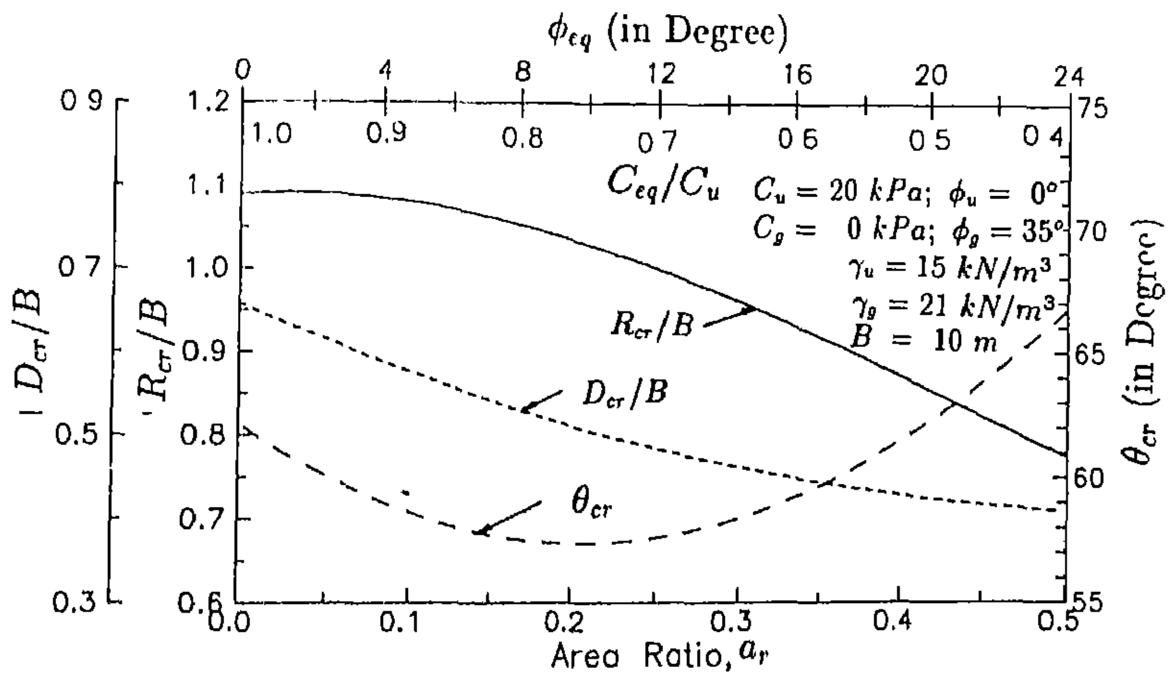

Figure 4.9: *Effect of Area Ratio on R_{cr} & θ_{cr}*

various values of the area ratio a_r, cohesion ratio, c_{eq}/c_u and equivalent angle of shearing resistance, ϕ_{eq}. The variation of the ratio of the depth of the critical slip circle to the width of the footing, D_{cr}/B is also plotted.

It is observed that the ratio, R_{cr}/B continuously decreases with increasing values of the area ratio, a_r, c_{eq}/c_u and ϕ_{eq}. The critical angle, θ_{cr} initially decreases for the value of the area ratio a_r upto 0.2 and then found to increase for $a_r \geq 0.2$

The depth ratio, D_{cr}/B decreases with increasing values of area ratio, a_r, indicating that with increasing degree of reinforcement, the critical slip surface becomes shallower. Keeping the other parameters constant, (shown in Fig 4.9), the radius of the critical slip surface is observed to be greater than the width of the footing for all values of area replacement ratio, a_r considered. The half the angle subtended at the center of the rotation (θ_{cr}) by the trial arc is observed to be ranging between 57° and 67°

4.5.2 Effect of $\frac{\gamma_{eq} \cdot B}{c_u}$ on R_{cr} and θ_{cr}

The effect of the non-dimensional term $\frac{\gamma_{eq} B}{c_u}$ on R_{cr} and θ_{cr} is depicted in Fig 4.10. It is observed that with increasing values of the factor $\frac{\gamma_{eq} B}{c_u}$, the ratio, R_{cr}/B continuously

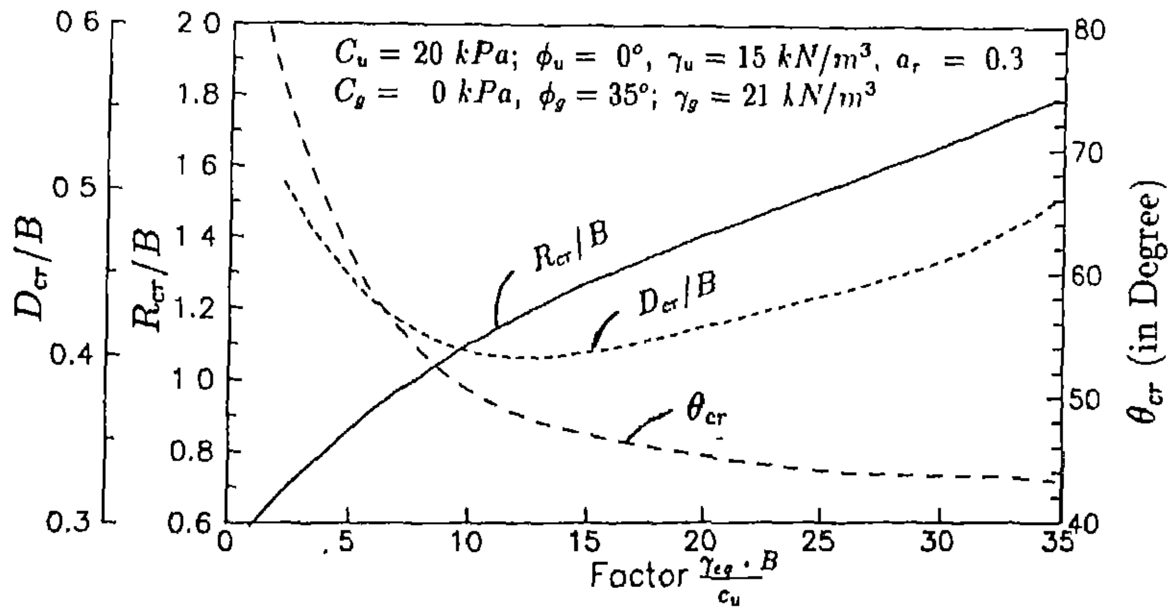

Figure 4.10: *Effect of* $\frac{\gamma_{eq} B}{c_u}$ on R_{cr} & θ_{cr}

increases from 0.64 to 1.80. For constant γ_{eq} & c_u, R_{cr} increases with increasing values of the width of the footing, B. This may be due to the assumption made regarding the slip circle that it starts from one of the edges of the footing.

The critical angle θ_{cr} is observed to be decreasing rapidly from 80° to 43° for the case considered. For $\frac{\gamma_{eq} \cdot B}{c_u} \geq 25$, the critical angle, θ_{cr} becomes almost constant equal to 43°

The depth ratio, D_{cr}/B initially decreases and then increases. This seems to be be due to the continuous increase in the value of the ratio, R_{cr}/B and decrease in the angle,

θ_{cr}. For the values of the factor $\frac{\gamma_{eq} B}{c_u}$ from 1.5 to 35.0, the ratio D_{cr}/B is observed to be in between 0.4 and 0.5.

4.5.3 Effect of c_u on BCR

The variation of bearing capacity ratio, BCR with area replacement ratio, a_r for the different values of undrained shear strength, c_u of the in-situ soil, and the factor $\frac{\gamma_u B}{c_u}$ is depicted in the Fig 4.11.

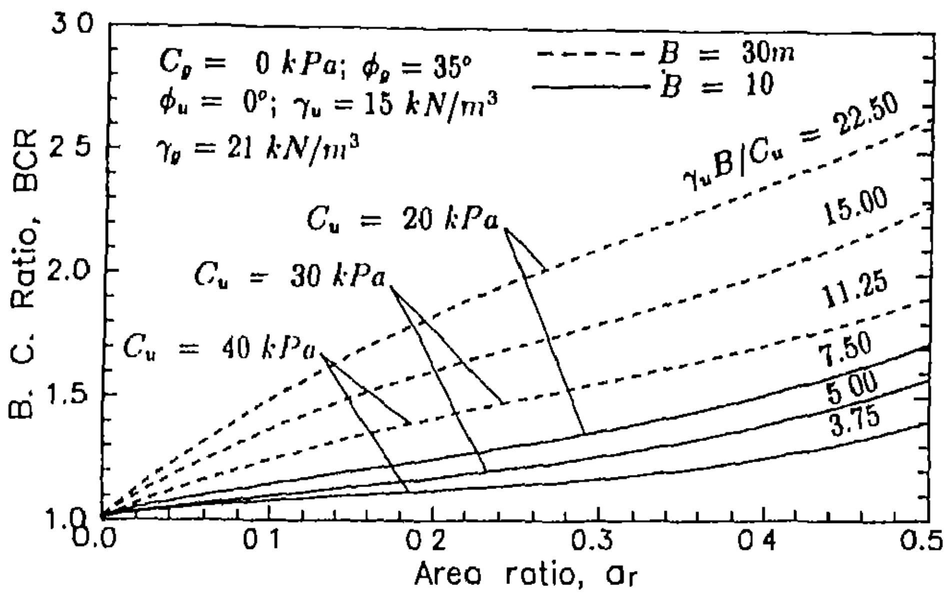

Figure 4.11: *BCR with Area Ratio - Effect of* c_u

It is observed that for the constant width of a footing, with increasing values of undrained shear strength, c_u, there is relatively lesser improvement in the bearing capacity of the composite soil. For B = 10 m and other parameters as shown in Fig. 4.11, the maximum values of the bearing capacity ratio, BCR are 1.72, 1.57 & 1.40 corresponding to c_u = 20, 30 & 40 kPa respectively. This may be due the reason that as c_u increases, the radius, R_{cr} of the critical slip circle decreases, leading to corresponding decrease in the length of the failure surface. In other words, the limit equilibrium approach predicts relatively higher values of improvement in the bearing capacity (BCR) for soft and very soft clays than that for the stiff and very stiff clays

4.5.4 Effect of ϕ_g on BCR

Fig. 4.12 shows the variation of BCR with area replacement ratio a_r for different values

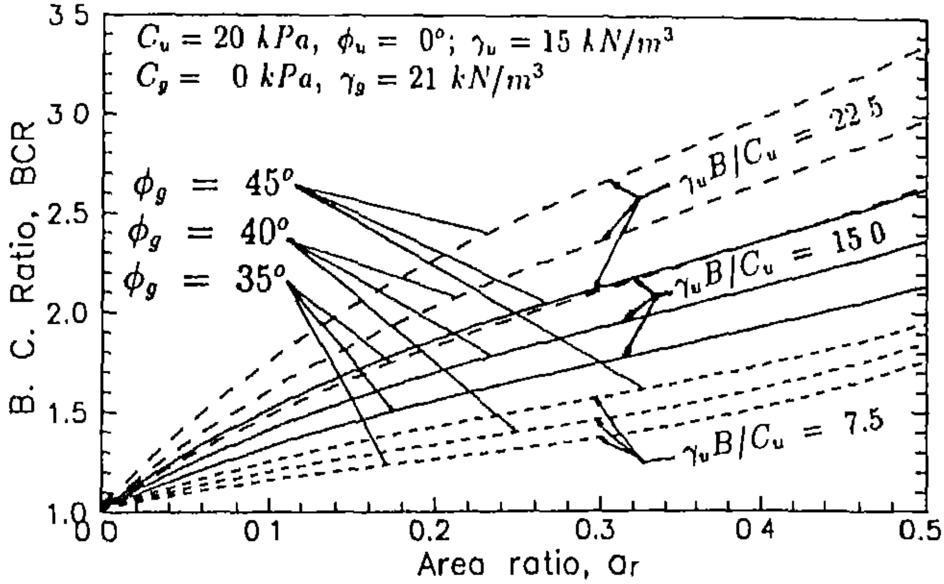

Figure 4.12: *BCR with Area Ratio - Effect of* ϕ_g

of angle of shearing resistance of the granular pile material, ϕ_g and the factor $\frac{\gamma_u B}{c_u}$.

For all values of area ratio, a_r considered, the bearing capacity ratio, BCR increases with increasing values of ϕ_g. This confirms the fact that the granular pile material having higher angle of shearing resistance, ϕ_g, results in higher bearing capacity of the composite ground, as the degree of reinforcement increases. All the curves show almost similar increase in the values of BCR. For the same angle of shearing resistance of the granular pile material, ϕ_g, with increase in the value of the factor $\frac{\gamma_u B}{c_u}$, there is increase in the bearing capacity ratio, BCR. The maximum BCR values for an area replacement ratio $a_r = 0.5$ and for the angle of shearing resistance of granular pile material, $\phi_g = 40°$, are 1.85, 2.35 & 3.00 corresponding to the values of the factor $\frac{\gamma_u B}{c_u} = 7.5$, 15.0 & 22.5 respectively.

4.5.5 Effect of $\frac{\gamma_u \cdot B}{c_u}$ on BCR

Fig. 4.13 shows the variation of the bearing capacity ratio, BCR with the factor, $\frac{\gamma_u \cdot B}{c_u}$ for different values of area replacement ratio, a_r. It is found that with increasing values of

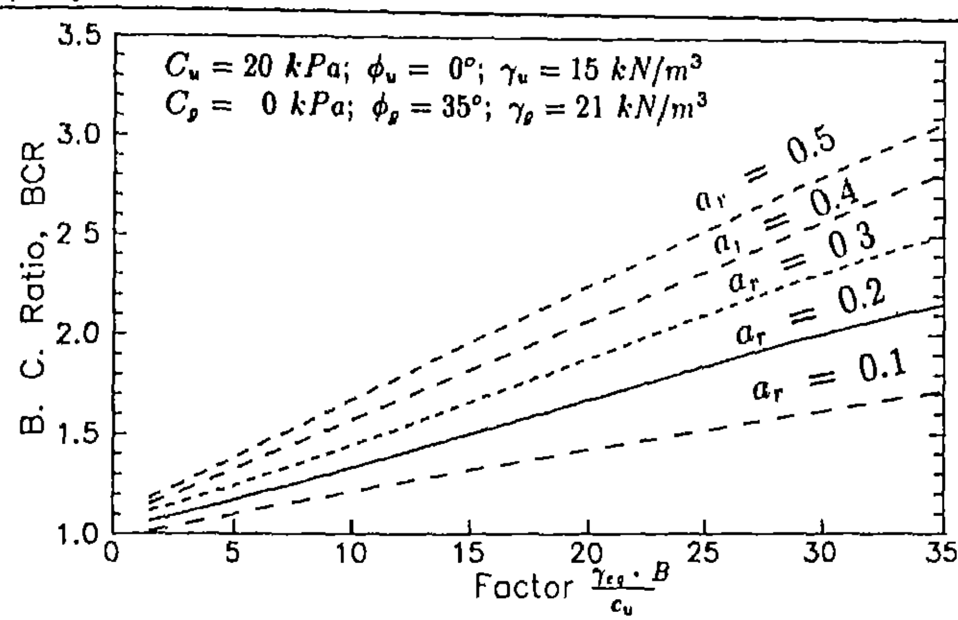

Figure 4.13: *Effect of* $\frac{\gamma_u B}{c_u}$ *on BCR*

the factor, $\frac{\gamma_u B}{c_u}$, the bearing capacity ratio, BCR linearly increases for all values of the area ratio, a_r considered.

As observed earlier, for constant undrained shear strength, c_u, with increasing the width of the footing, B, there is an increase in the radius of the critical slip circle, R_{cr} and in the corresponding length of the slip surface This results in the higher values of bearing capacity ratio, BCR. For constant width of the footing, B, with increase in the value of undrained shear strength, c_u, there is relatively less improvement in the bearing capacity of such reinforced soil.

4.6 Conclusions

Based on the study presented in this Chapter, the following conclusions can be drawn:

- For the given parameters of in-situ soil, granular pile material and area ratio, the method suggested by Enoki *et al.* (1991) is the most appropriate method to determine the equivalent soil parameters for both types of analysis (undrained and drained).

- The two approaches viz., approximate lower bound solution and the limit equilibrium method presented in this Chapter, can be used to determine the bearing capacity of a soil reinforced with a group of granular piles.

- The approximate lower bound solution gives linear increase in the bearing capacity ratio for all of values of area ratio considered.

- The limit equilibrium approach is more appropriate to use than the approximate lower bound method as it involves the determination of the critical circular failure surface and takes into account the effect of the width of the footing.

- By increasing the width of the footing, the bearing capacity ratio increases The limit equilibrium approach predicts relatively higher values of improvement in the bearing capacity (BCR) for soft and very soft clays than that for the stiff and very stiff clays

- As all of these approaches predict lower values of the bearing capacity ratio than actually observed values, modification by using upper bound solution or using different shapes of failure surface (e. g log spiral) is necessary

Chapter 5

SLOPE STABILITY

5.1 Introduction

Use of granular piles is one of the preferred alternatives to improve marginal sites involving soft soils, to permit construction of embankments such as for highways or for bridge approaches. Granular piles increase the factor of safety to an acceptable level with respect to general rotational or translational failures, to an acceptable level by forming reinforcing elements. Granular piles under the embankments are also used to reduce deformations Generally the whole area under the embankment is treated by adopting an area ratio in the range 10 – 40%. However granular piles can also be used to increase the stability of existing slopes undergoing creep (Rathgeb & Kutzner, 1975) In this case only the area under side slope is treated using higher area replacement ratio Hence for the design of embankment constructed on soft natural soil reinforced with granular piles, stability analysis is an important step.

5.2 Stability of Composite Ground

Stability analysis of reinforced ground using granular piles is usually carried out in the same manner as for normal slope stability problem except that the effect of stress concentration on granular pile is considered. Based on the undrained shear strength of soft soil, the transverse shear strength of granular pile and area replacement ratio, shear strength of composite ground is determined. Using elasticity and Rankine's earth pressure theory, Priebe (1976) has presented a method to determine the shear strength parameters (cohesion and angle of shearing resistance) for composite ground which then can be used

in the conventional method of stability analysis to determine the factor of safety. The method given by Aboshi *et al.* (1979) determines the weighted average shear strength of granular pile and surrounding clay assuming appropriate value of stress concentration factor. Barksdale and Bachus (1983) list further two approaches such as - Profile method and Lumped Mass Moment method In Profile method the effect of stress concentration is handled by taking thin, fictitious soil layer at the embankment-ground interface Lumped mass moment method is generally suitable for hand calculation (Bergado *et al*, 1991). Based on direct shear tests on model unit cell, Madhav (1992) reported that shear strength of composite ground also depends upon the stress level and the modular ratio of granular pile and the surrounding soft soil.

It is observed (Aboshi *et al*, 1979, Rathgeb & Kutzner, 1975) that stability analysis of embankments constructed over soft ground reinforced with granular piles is generally carried out with circular slip surfaces Based on Janbu's generalised procedure of slices, Sabhahit (1994) has carried out stability analysis using non-circular failure surfaces and reported that circular slip surface analysis based on Fellenius method (Rathgeb & Kutzner, 1975) overestimates the factor of safety by 4.0% only. He also pointed that there exists an optimum pile length beyond which the factor of safety is unaffected by granular pile length for given embankment geometry, embankment and foundation soil properties

In the present analysis, the shear strength of improved ground is estimated based on equivalent anisotropic strength concept given by Enoki *et al* (1991) It is found that the earlier studies do not indicate the required extent of treatment of foundation soil for optimal increase in the factor of safety of the embankment founded on it An attempt is made to study the effect of varying of the extent of treated ground beyond the toe or towards the slope on the factor of safety, to help the designer in the choice of the appropriate design parameters

5.3 Method of Analysis

Under limit equilibrium theory, although several stability analyses methods have been developed, simplified Bishop's method which satisfies the moment equilibrium is considered for the present analysis. Since Sabhahit (1994) reports that factors of safety obtained by circular and non-circular slip surfaces do not vary significantly; the stability analysis is carried out by using circular slip surfaces only.

5.4 Problem Formulation

Simple slopes without side berms are considered for the present analysis. Fig 5.1 shows an

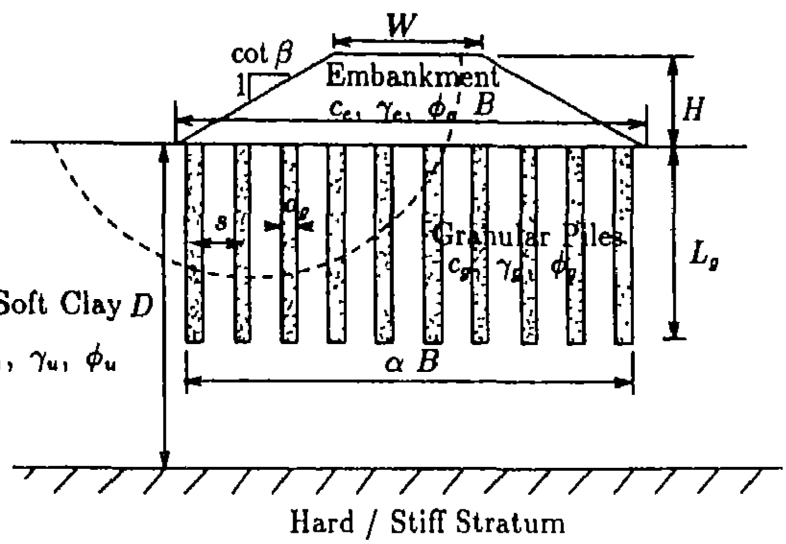

Figure 5.1: *Embankment on Reinforced Ground*

embankment having height, H, crest width, W, base width, B and side slopes 1V.cotβ H constructed on soft ground. Embankment material has unit weight γ_e, cohesion, c_e and angle of shearing resistance, ϕ_e while the soft natural soil has unit weight, γ_u, cohesion, c_u and frictional angle, ϕ_u. The soft clay having thickness, D, is underlain by a relatively stiff/hard stratum.

The clay deposit is reinforced with granular piles of length, L_g, (for end bearing piles $L_g = D$) having diameter, d_g and spacing, s. Depending upon the pattern of piles, area ratio can be determined as

$$a_r = c_s \, (d_g \, / \, s)^2$$

where, $c_s = 1.27$ for triangular; and

$= 1.013$ for square patterns.

The extent of treated ground is denoted by $\alpha\ B$ Hence if

$\alpha = 1$; treated area is upto toe;

< 1 ; treated area is within embankment;

> 1 ; treated area extends beyond the toe.

As granular piles reinforce the ground beneath the structure, they (granular piles and the ground) should be treated as an entity and the focus of analysis should lean towards

the global response of the composite ground rather than towards the detailed response of each reinforcing element (Randolph, 1994).

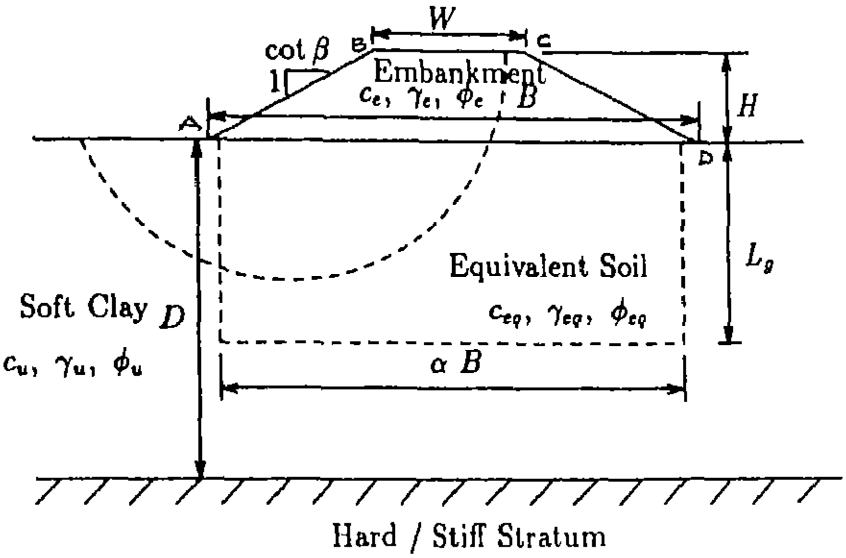

Figure 5.2· *Replacement by an Equivalent Soil*

The treated ground is replaced by an equivalent soil as shown in Fig. 5 2 with unit weight, γ_{eq}, cohesion, c_{eq} and angle of frictional resistance, ϕ_{eq}, determined as explained in Section 4.3.1

5.4.1 Determination of Factor of Safety

Trial failure circular surfaces are generated from left to right. Initial point of a slip surface is taken at or left of point A (Fig. 5.2) and ending point anywhere between the left crest and the right toe (B-C-D). Slip surfaces located above the hard stratum and within the clay only are considered. For a given set of properties, the factor of safety is obtained using "PCSTABL5" (Carpenter, 1986) program For each case, 100 trial surfaces were generated by varying their locations

5.4.2 Comparison of Present Approach

To validate the results obtained by the present method of analysis, the stability of an embankment as reported by Rathgeb & Kutzner (1975) has been taken up and studied. The concerned embankment was 11.4 m high constructed on soft clay having thickness of

6.5 m and underlain by loamy gravel. Other geotechnical properties are shown in Fig. 5.3

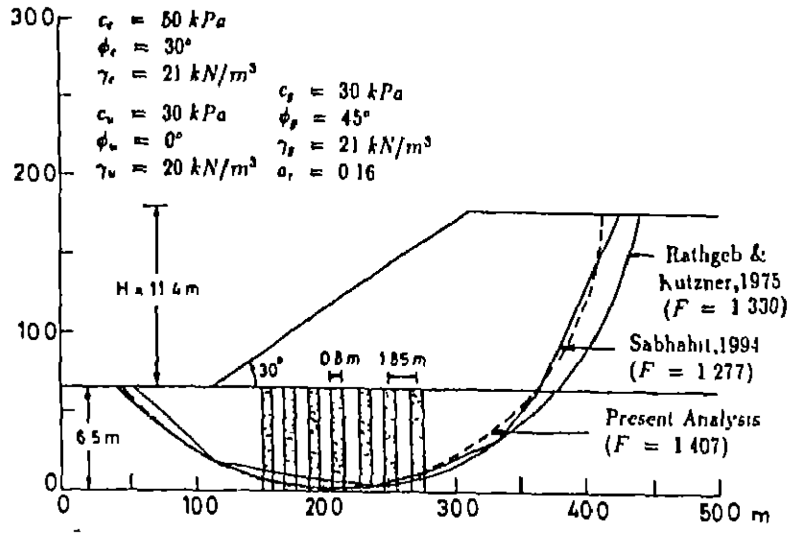

Figure 5.3 *Comparison of different methods of stability analysis*

The reported (Rathgeb & Kutzner, 1975) factor of safety is 1.03 for the unreinforced case and 1.33 for the improved soil corresponding to an area ratio a_r = 0.16 with the critical slip surface touching the hard stratum. Sabhahit (1994) predicted a factor of safety of 0.99 for untreated soil and 1.277 for the reinforced soil. The present analysis predicts factor of safety of 0.961 for the unreinforced case and 1.469 by Janbu's method (random slip surface); 1.407 by Janbu's method (circular slip surface) and 1.458 by modified Bishop's analysis (circular slip surface). Thus the present method of analysis predicts factors of safety that are close to those reported earlier (Rathgeb & Kutzner, 1975) and obtained (Sabhahit, 1994) for the unreinforced case but some what higher for the reinforced case

5.4.3 Parametric Study

Results to show the influence of different parameters on the stability of the considered embankment on reinforced soft soil are presented based on an equivalent anisotropic shear strength concept. As the diameter, spacing and pattern of granular piles can be conveniently studied by just considering the area ratio (a_r), the same is considered in the present analysis. The various design parameters considered are the width and the height of the embankment, side slopes, embankment cohesion and angle of shearing resistance,

undrained shear strength of foundation soil, shear parameters of granular pile, thickness of clay layer and length of granular piles. It is assumed that the granular pile material has no cohesion and the foundation soil is purely cohesive. The extent of treated ground is expressed in terms of the base width (B) of the embankment as αB. The length of granular pile is only to provide the stability for the embankment but not on consideration to reduce the settlement. These parameters are varied and results are expressed in terms of factor of safety. Once the area ratio is known, the spacing and diameter of granular piles can be determined based on equipment available. These graphs can also be used for design of a given embankment.

The various values of the parameters considered for

the Embankment:

- Height, H = 8 m;
- Crest Width, W' = 4, 12, 20 m,
- Base Width, B = 36, 44, 52 m,
- Side Slopes: 1 H : cot β V where cot β = 1.5, 2.0, 2.5,
- Unit Weight, γ_e = 21 kN/m^3;
- Cohesion, c_e = 20, 25, & 30 kPa;
- Angle of shearing resistance, ϕ_e = 20^o, 25^o, 30^o

the Soft Soil:

- Unit Weight, γ_u = 19 kN/m^3;
- Undrained cohesion, c_u = 15, 20, 25 kPa (average values) and varying with depth (as shown in Fig. 5.6.b);
- Angle of shearing resistance, ϕ_u = 0^o;
- Depth of clayey bed, D = 8, 10, 12, 14, 16, 20, 24, 30 m.

the Granular Pile:

- Unit Weight, γ_g = 21 kN/m^3;
- Cohesion of granular pile material, c_g = 0 kPa;
- Angle of shearing resistance, ϕ_g = 35^o, 40^o & 45^o;
- Length of granular piles, L_g = 8, 9, 10, 11, 12, 13, 14 m;
- Area ratio, a_r = 0.0, 0.1, 0.2, 0.3, 0.4 & 0.5.

For the extent of treated ground, the factor α is varied as,

α = 0.7, 0.8, 0.9, 1.0, 1.1, 1.2, 1.3, 1.4 & 1.5

The variation of factor of safety with area replacement ratio for different values of factor α is shown in Fig 5.4, keeping all the other variables constant. The factor α is the

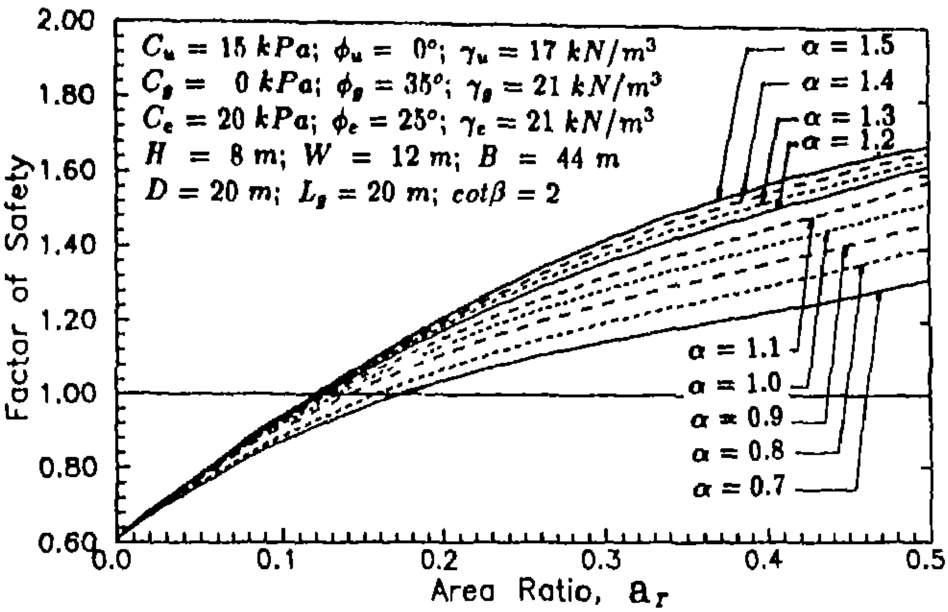

Figure 5.4: *Factor of Safety with Area Ratio for various values of α*

ratio of the width of treated ground to the base width of the embankment and indicates the extent of ground strengthened by granular piles. For all values of α considered, the factor of safety increases with increasing values of area ratio. For a particular value of area ratio, the minimum area ratio, to obtain the factor of safety at least equal to unity, is observed to be in the range of 12 – 17% for α varying from 1.5 to 0.7. At a_r = 0.35, the increments in factor of safety are almost constant and equal to 0.25 for increase in values of α from 0.7 to 1.2. However when α increases from 1.2 to 1.5, the factor of safety increases from 1.44 to 1.52 only.

To show quantitatively the effect of the width of treated ground on the factor of safety, percentage increase in factor of safety with respect to unreinforced case has been estimated for different values of area ratio, a_r and presented in Fig 5.5. For the range of area ratios considered, improvement in factor of safety can be achieved by increasing the value of factor α. For low area ratios, (a_r = 0.1 to 0.2), the improvement in factor of safety is gradual for all values of α considered. For higher area ratios, (a_r > 0.3), in the range of α varying from 0.7 to 1.2, percentage improvement in factor of safety increases linearly and

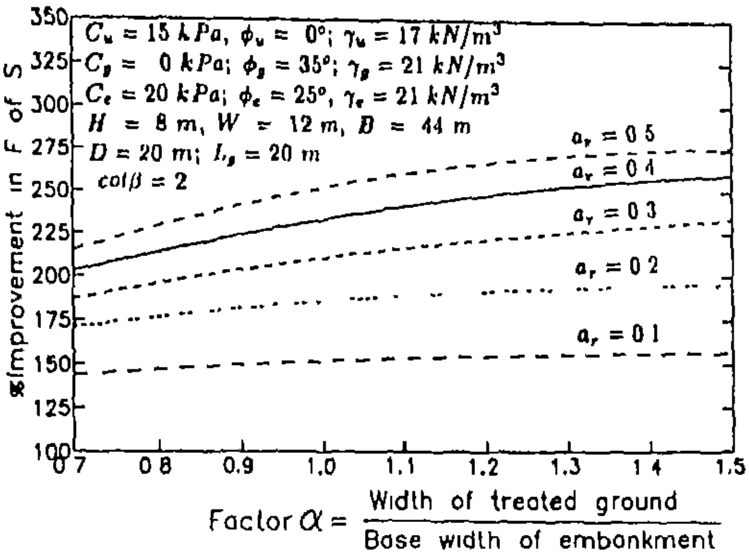

Figure 5.5. *% Improvement in Factor of Safety with Area Ratio*

becomes almost constant beyond $\alpha = 1.2$ indicating that there is no additional benefit if width of reinforced ground is greater than 1.2 times the base width of the embankment

Effect of depth of hard stratum (D = 8, 10, 12, 14, 16, 20, 24, 30 m) on factor of safety is plotted in Fig 5.6 with end bearing granular piles It is interesting to note that

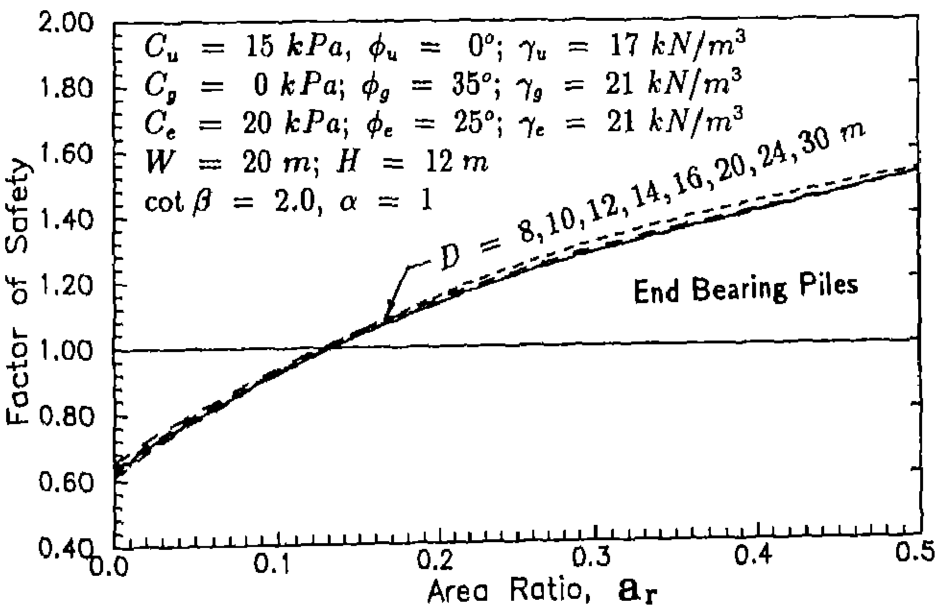

Figure 5.6: *Factor of Safety with Area Ratio – effect of D*

for all values of area ratio considered, the factor of safety is nearly independent of depth of hard stratum. It is found that the critical slip surfaces do not vary with D for all a_r

and the maximum depth of the critical slip surface is found to be less than 8 m (the least depth considered) for the cases considered.

However length and diameter of granular piles are very important parameters, considering economy. Increase in pile diameter is cost effective as compared to increase in pile length. In practice length of granular piles is provided upto the hard stratum if it is available at shallow depth and termed as end bearing piles. Whereas if thickness of soft clayey bed is very high, then floating granular piles are to be designed for an economical length and provided. An attempt has been made here to investigate the minimum required length of granular piles when hard stratum is available at a great depth.

Fig. 5.7 shows the variation of factor of safety with area ratio for lengths of granular

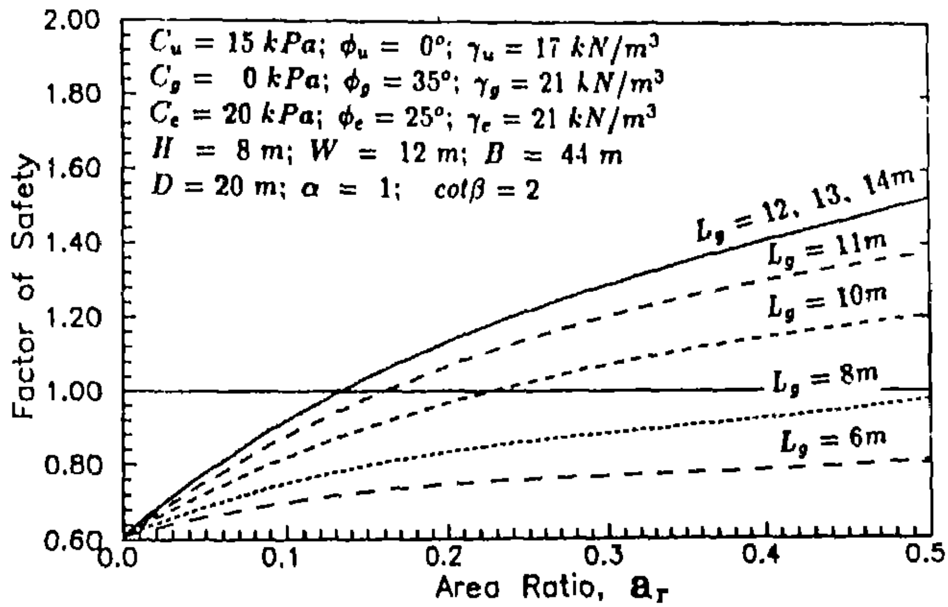

Figure 5.7: *Factor of Safety with Area Ratio - effect of L_g*

piles, $L_g = 6, 8, 10, 11, 12, 13,$ *and* $14\ m$ and for thickness of soft clay layer, $D = 20\ m$. For all values of L_g considered, factor of safety increases, with increase in area ratio. However for $L_g = 6\ \&\ 8\ m$, the maximum value of factor of safety is less than 1.0 even with an area ratio of 0.5. This is due to major portion of the critical slip surface passes through unreinforced ground than through the treated ground, as the depth of the critical slip surface is observed to be greater than the length of granular piles provided. For the pile lengths beyond 12 m, the factor of safety remains constant indicating that $L_g = 12\ m$ is the optimum pile length for the case considered and there is no change in

the locations of critical slip surfaces. The minimum area ratios for factor of safety to be greater than 1.0, are in the range of 0.13 to 0.23 for $L_g = 12\ m$ to $10\ m$ respectively.

From Fig 5.8 it is observed that for thickness of clayey layer, D beyond 12 m, the optimum pile length ($L_g = 12\ m$) remains constant for this given embankment geometry and all other parameters as shown in Fig 5.8. For shallow depth of hard stratum i.e

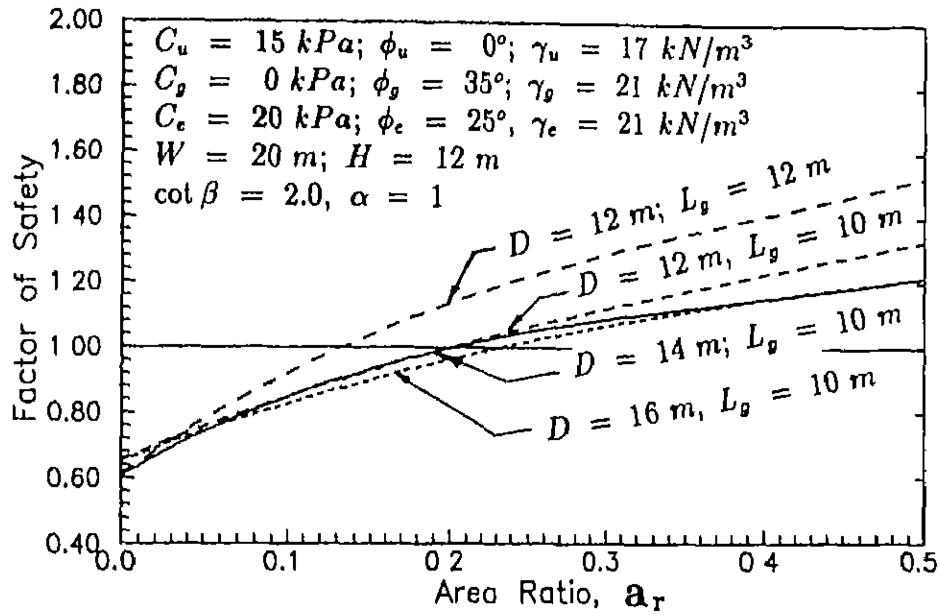

Figure 5.8: *Factor of Safety with Area Ratio for different D & L_g*

$D < 12\ m$, end bearing piles should be provided for the maximum increase in factor of safety

The variation of factor of safety with area ratio for $D = 12\ m$ & $L_g = 10\ m$, $D = 14\ m$ & $L_g = 10\ m$, and $D = 16\ m$ & $L_g = 10\ m$ are also plotted in Fig.5.8 For all values of area ratio, the factor of safety for $D = 12\ m$ & $L_g = 10\ m$ are higher than those for $D = 16\ m$ & $L_g = 10\ m$. For low area ratios ($a_r < 0.2$), factor of safety obtained for $D = 14\ m$ & $L_g = 10\ m$ are identical with those for $D = 12\ m$ & $L_g = 10\ m$. However as the foundation soil gains significant strength due to higher area ratio, and critical slip surface moves downwards resulting in same critical slip surface as those for $D = 16\ m$ & $L_g = 10\ m$ giving low factors of safety.

The variations of factor of safety with area ratio for various values of side slopes and base widths ($\cot\beta = 1.5$, 2.0, 2.5 and $B = 36$, 44, 52 m) are plotted in Fig. 5.9 for constant top crest width, $W = 12\ m$ and optimum pile length, $L_g = 12\ m$ All curves show similar variation in factor of safety with increasing values of area ratio. By

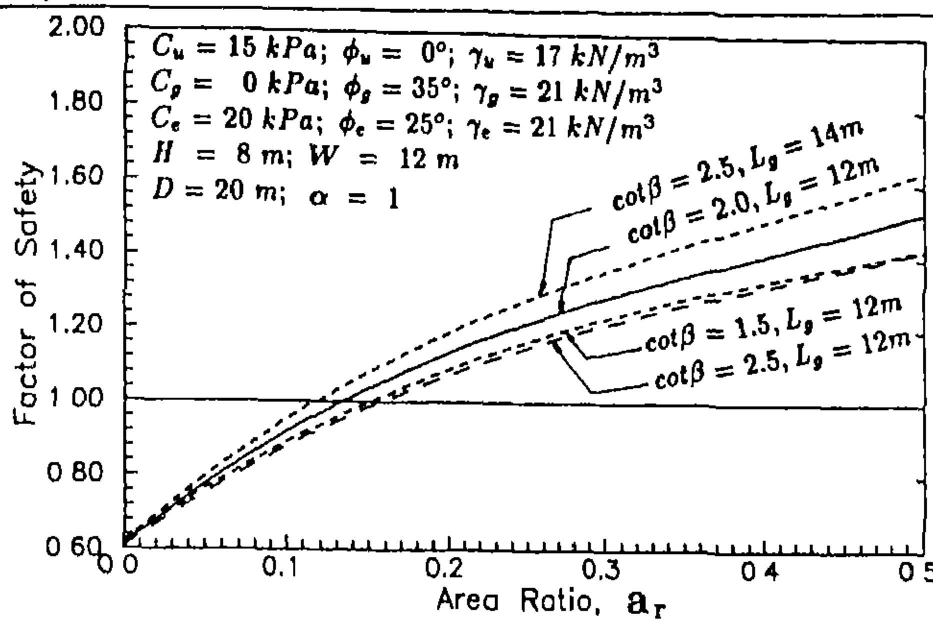

Figure 5.9 *Factor of Safety with Area Ratio for different Side Slopes & Base Width*

increasing value of $\cot\beta$ from 1.5 to 2.0, (flattening side slopes), factor of safety increases for all values of area ratio. However for $\cot\beta = 2.5$ & $L_g = 12\ m$, the factors of safety obtained are less than those for $\cot\beta = 2.0$ & $L_g = 12\ m$ for all values of a_r. To understand this behaviour, the critical slip surfaces for $a_r = 0.3$ are determined and plotted in Fig. 5.10.

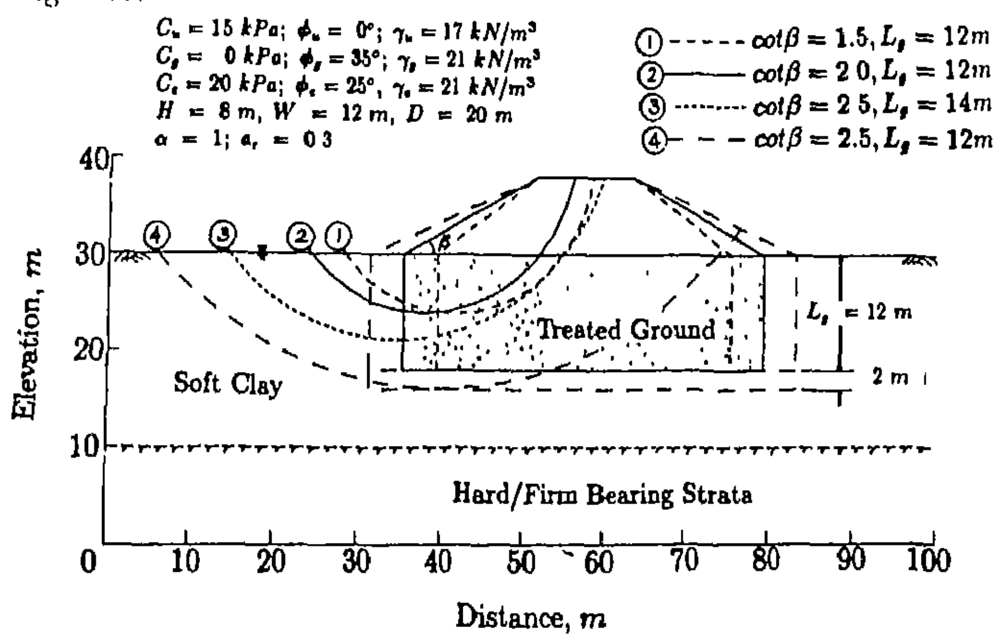

Figure 5.10: *Critical Slip Surfaces for different Side Slopes & L_g*

It can be seen that for $\cot\beta = 2.5$ & $L_g = 12\ m$ the critical slip surface becomes very deep and since its major portion passes through soft clay (slip surface 4) low value

of factor of safety is obtained. However by increasing L_g by 2 m only, the critical slip surface moves upwards (slip surface 3) resulting in higher factor of safety

The variation of factor of safety with area ratio for different values of the side slopes and crest width ($\cot\beta$ = 1.5, 2.0, 2.5 & W = 20, 12, 4 m) are plotted in Fig 5.11 for B = 44 m & L_g = 12 m All curves show similar increase in factor of safety with

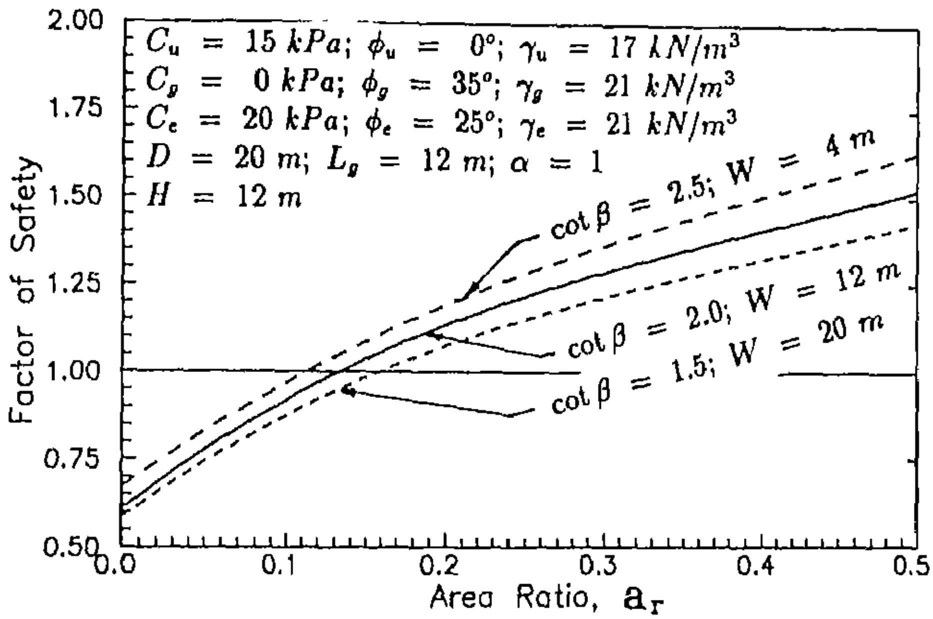

Figure 5.11 *Factor of Safety with Area Ratio for different Side Slopes & Crest Width*

increase in area ratio By increasing values of $\cot\beta$, i.e. flattening the side slopes, the factor of safety increases as the magnitude of driving moment decreases. However there is decrease in top crest width as the base width is kept constant for change in value of $\cot\beta$.

The variation of factor of safety with area ratio for different values of the undrained shear strength of the in-situ soil is plotted in Fig 5.12. Three average undrained shear strengths as 10 kPa, 15 kPa & 20 kPa and one which varies with depth (as shown in Fig. 5.7.b) are considered. As expected, the factor of safety increases with increasing values of area ratio for all values of c_u. Using average shear strength of $c_u = 10$ kPa, for given thickness of clay layer, the factors of safety obtained are less than those obtained using the actual in-situ shear strength profile whereas use of average shear strength, c_u = 20 kPa leads to overestimation of factor of safety. Hence it would be more appropriate to determine the factor of safety based on the actual in-situ shear strength profile than the average shear strength values.

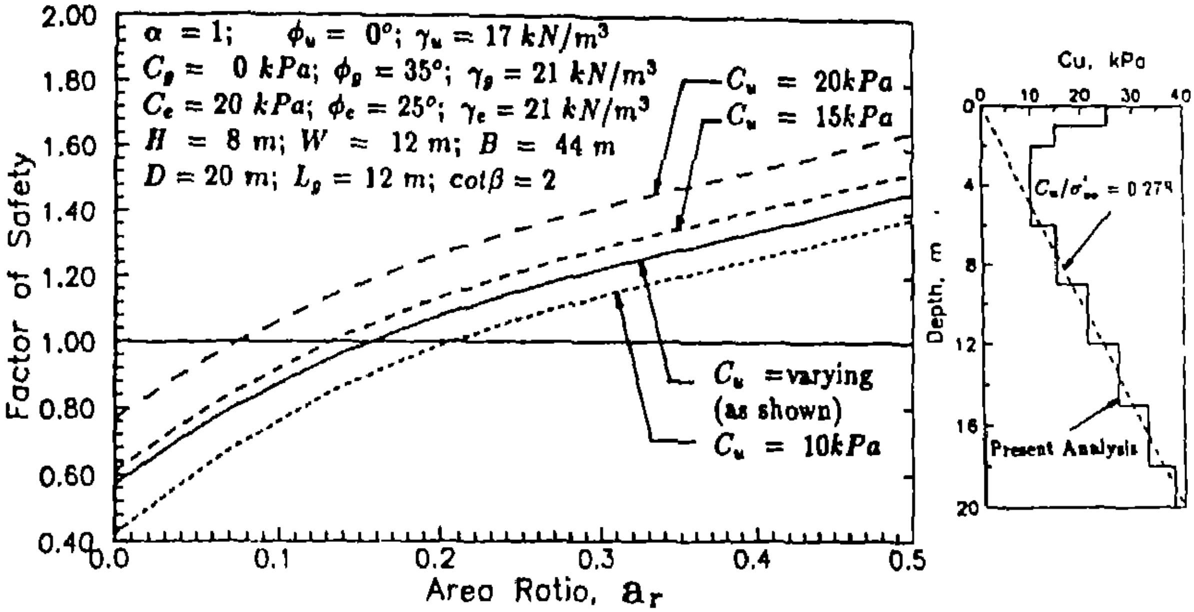

Figure 5.12: *Factor of Safety with Area Ratio – effect of c_u*

The variation of factor of safety with area ratio for various values of angles of shearing resistance of granular pile material is shown in Fig. 5.13. All curves show similar variation

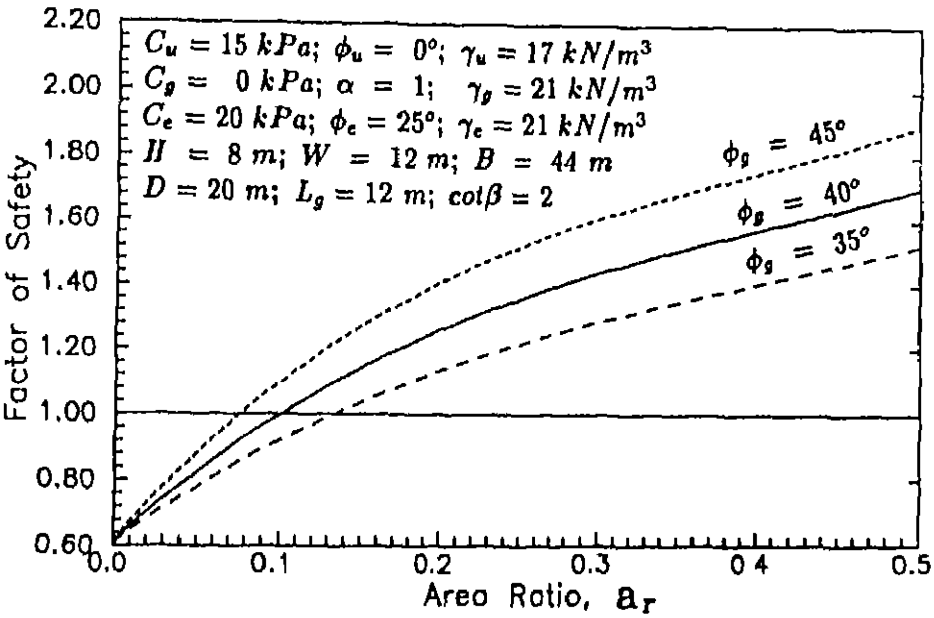

Figure 5.13: *Factor of Safety with Area Ratio – effect of ϕ_g*

of factor of safety with area ratio. For all values of ϕ_g considered, factor of safety increases non-linearly for lower area ratios but the increase in factor of safety is observed to be linear for higher area ratios. The minimum area ratio required for factor of safety > 1.0, drops from 14% to 8% when ϕ_g increases from 35^o to 45^o. For $a_r = 0.3$, the factors of safety are 1.28, 1.44, & 1.60 corresponding to $\phi_g = 35^o$, 40^o & 45^o respectively showing equal

increments in factor of safety.

Effect of angle of shearing resistance (ϕ_e = 20°, 25°, 30°) and cohesion (c_e = 20, 25, 30 kPa) of embankment material on factor of safety are shown in Fig 5.14 and Fig 5.15.

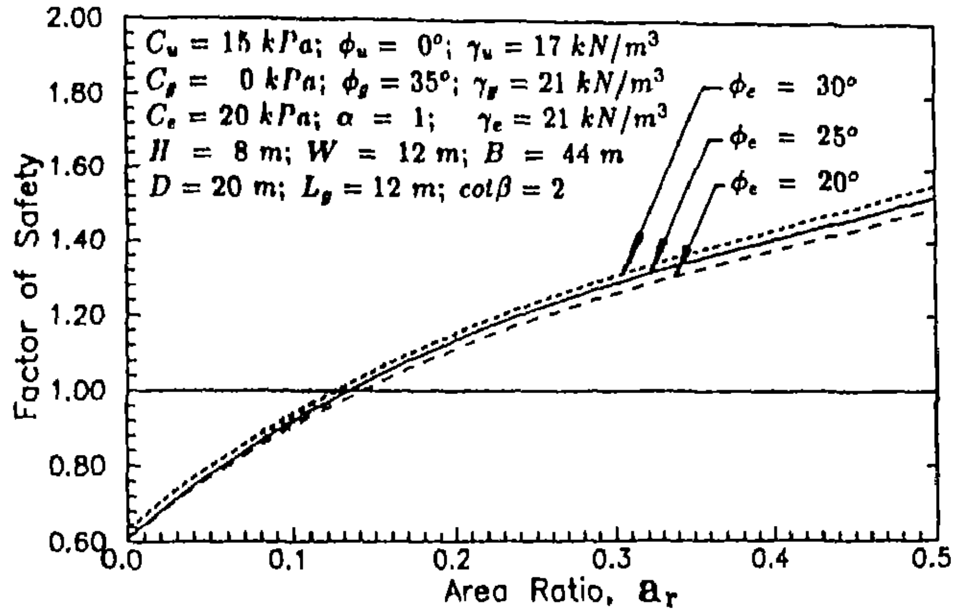

Figure 5.14. *Factor of Safety with Area Ratio – effect of* ϕ_e

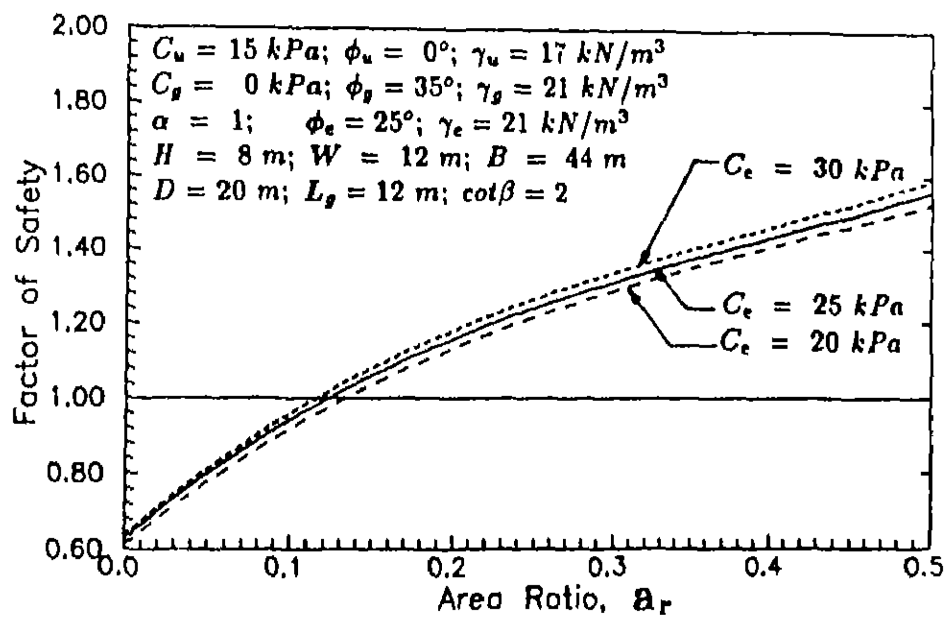

Figure 5.15: *Factor of Safety with Area Ratio – effect of* c_e

For both angle of shearing resistance and cohesion of embankment material, factor of safety increases non-linearly for low values of area ratio but the increase in factor of

safety is observed to be linear for higher values of area ratio. However there is no marked increase in factor of safety for increasing either $\phi_c = 20^o$ to 30^o or $c_c = 20\ kPa$ to $30\ kPa$ for all values of area ratio considered. Hence the stability of embankment constructed on soft ground reinforced by granular piles is mainly governed by the strength properties of unreinforced ground and granular pile material

5.5 Conclusions

Based on the results presented in this study, the following conclusions can be drawn.

- The analysis procedure presented for embankments constructed over a soft foundation soils reinforced with granular piles, is efficient to estimate the factors of safety
- The factor of safety increases with increase in the area ratio of replacement
- Optimal improvement in the factor of safety of the embankments is achieved by providing width of reinforced zone 1.2 times the base width of embankment.
- For the cases considered, $H/D = 1.0$ to 3.75, the factor of safety is observed to be independent of thickness of clayey bed for end bearing piles.
- For values of H/D upto 1.75, end bearing piles can be provided while for higher values of H/D, desired level of factor of safety can be achieved by providing floating piles having length $L_g = 1.75H$
- The geometry of embankment (side slopes, crest width and base width) has major influence on factor of safety.
- Factor of safety increases with increasing values of undrained shear strength of foundation soil and angle of shearing resistance of granular pile material However it will be more appropriate to use actual in-situ shear strength profile rather than average values of strength throughout the depth of clayey bed
- Stability of embankments constructed on reinforced ground using granular piles is governed by properties of in-situ soil and granular pile material but independent of the properties of embankment material.

Chapter 6

CONCLUDING REMARKS

6.1 Conclusions

Based on the study presented in the previous Chapters, the following conclusions can be drawn:

6.1.1 Load Tests

- ◇ For model load tests, *reconsolidation technique* offers the best choice to prepare small scale, uniform and identical soil deposits in the laboratory with controlled stress history. Further soil properties are reproducible.
- ◇ Large volume change taking place during the consolidation phase can be avoided by keeping low initial water content. However, it should be ensured that slurry is well mixed to produce homogeneous deposits.
- ◇ The amount of time required for the consolidation of soil deposits can be approximately estimated by using the criteria suggested by McManus & Kulhawy (1993).
- ◇ Due to dry installation method of granular piles, surface heave and enlargement of uncased holes were observed.
- ◇ From the results of load tests it is observed that with increase in area ratio, the subgrade modulus, ultimate bearing capacity and undrained modulus increase.

- For same area ratio, the values of subgrade modulus, ultimate bearing capacity and undrained modulus from the load tests on plate with single granular pile are greater than those for the load tests with group of three granular piles.

- Subgrade modulus of the granular pile reinforced soil is observed to decrease with increase in plate diameter.

- Bearing capacity ratio from load tests on plate with single granular pile is less than the field observed and suggested values for low area ratio However from the results of load tests with group of three piles, the BCR values are in the range of actually observed and suggested values.

- Very small increase in undrained modulus is observed for all the load tests

- The observed bulging of granular piles near the top surface at depth of approximately equal to one to one-half of pile diameter is similar to the reported observations

6.1.2 Bearing Capacity

- The two approaches viz , approximate lower bound solu tion and the limit equilibrium method can be used to determine the bearing capacity of a soil reinforced with a group of granular piles.

- The approximate lower bound solution gives linear increase in the bearing capacity ratio for all of values of area ratio considered Further, the limit equilibrium approach is more appropriate to use than the approximate lower bound method as it involves the determination of the critical circular failure surface and takes into account the effect of the width of the footing.

- By increasing the width of the footing, the bearing capacity ratio increases The limit equilibrium approach predicts relatively higher values of improvement in the bearing capacity (BCR) for soft and very soft clays than that for the stiff and very stiff clays.

6.1.3 Slope Stability

- The analysis procedure presented for embankments constructed over a soft foundation soils reinforced with granular piles, is efficient to estimate the factors of safety.

- Optimal improvement in the factor of safety of the embankments is achieved by providing width of reinforced zone 1.2 times the base width of embankment.

- For the cases considered, $H/D = 1.0$ to 3.75, the factor of safety is observed to be independent of thickness of clayey bed for end bearing piles

- For values of H/D upto 1.75, end bearing piles can be provided while for higher values of H/D, desired level of factor of safety can be achieved by providing floating piles having length $L_g = 1.75H$.

- The geometry of embankment (side slopes, crest width a nd base width) has major influence on factor of safety. However, it is observed that stability of embankments is not governed by the strength properties of embankment material

6.2 Suggestions for Further Work

For preparation of soil deposits using reconsolidation technique, loading by means of hydraulic jack will be more appropriate to aviod stress fluctuations. Further, use of pore pressure measurement devices will reflect more detailed consolidation behaviour and strength properties.

Detailed load testing program can be carried out to study effect of preconsolidation stress on behaviour of granular pile reinforced soil.

As all of these available approaches predict lower values of the bearing capacity ratio than actually observed values, modification by using upper bound solution or using different shapes of failure surface (e. g. log spiral) is necessary.

The present approach for stability of embankments on granular pile reinforced ground can be modified to take into account the effect of stress concentration in the granular piles.

References

Aboshi, H. (1979) "The Compozer, A Method to Improve characteristics of Soft Clays by Inclusion of large diameter Stone Columns" *Int Conf. on Soil Reinforcement*, Paris, Vol. 1, pp. 211–216.

Aboshi, H. & Suematsu, N. (1985) "Sand Compaction Pile Method. State-of the-Art Paper." *Proc. 3rd Int. Geotechnical Seminar on Soil Improvement Methods*, Nanyang Technical Instiute, Singapore.

Alamgir, M. Miura, N., & Madhav, M R. (1994). "Analysis of Granular Column Reinforced Ground–II: Stress Transfer from Granular Column to Soil" *Report of the Faculty of Science & Engineering*, Saga University, Vol. **23**, pp 81–94.

Almedia, M. S. S., Davies, M. C. R & Parry, R. H G. (1985). "Centrifuge Tests of Embankments on Strengthened and Unstrengthened Clay Foundations." *Géotechnique*, Vol.**35**(4), pp. 425–441.

Asaoka, A., Matsuo, M. & Kodaka, T. (1994). "Bearing Capacity of Clay Improved with Sand Compaction Piles." *Proc. XIIIth Int Conf. on Soil Mechanics & Foundation Engineering*, New Delhi, pp. 763–766

Baez, J. I. & Martin, G. R. (1992). "Quantitative Evaluation of Stone Column Techniques for Earthquake Liquefaction Migitation" *Proc Xth World Conf. Earthquake Engineering*, Madrid, Vol. 3, pp. 1477–1483.

Barksdale, R. D. & Bachus, R. C. (1983). "Design and Construction of Stone Columns." Report No. FHWA/RD-83/026, National Information Service, Springfield, Virginia, U.S.A.

Bergado, D.T., Rantucci, G. & Widdodo, S. (1984) "Full Scale Load Tests on Granular Piles and Sand Drains in the Soft Bangkok Clay." *Proc. Int. Conf on In-situ Soil & Rock Reinforcement*, Paris, pp 111-118.

Bergado, D.T., Alfaro, M.C & Chai, J.C (1987) "Full Scale Load Test of Granular Piles with Different Densities and Different Proportions of Gravel and Sand in Soft Bangkok Clay." *Soils and Foundations*, Vol.**27**, No 1, pp. 86-93

Bergado, D. T., Alfaro, M. C. & Chai, J. C. (1991). "The Granular Pile Its present state & future prospects for Improvement of Soft Bangkok Clay " *Geotechnical Engineering*, No. **22**, pp. 143-147.

Bouassida, M., de Buhan, P. & Dormieux L (1995). "Bearing Capacity of a Foundation resting on a Soil Improved by a Group of Columns " *Géotechnique*, Vol.**45**(1), pp. 25-34.

Carpenter, J. R. (1986). "STABL5/PCSTABL5 User Manual " Research Report No JHRP-86-14. Engineering Experiment Station, Purdue University

Clency, P. & Randolph, M. F. (1993). "An Approximate Analysis Procedure for Piled Raft Foundations." *International Journal for Numerical & Analytical Methods in Geomechanics*, Vol. **17**, pp. 849-869.

Datye, K. R. & Madhav, M. R. (1988). "Case Histories of Foundations with Stone Columns." *Proc. Int. Conf. on Case Histories in Geotechnical Engineering*, St. Louis, Paper No. 537, pp. 1-12.

DeStephen, R. A., Kozera, D. W. & Swekosky, F. J (1993). "Design of Floating Stone Columns in Hydraulic Fill." *Proc. Conf. on Grouting / Soil Improvement and Geosynthetics*, Geotechnical Special Publication, No. 30, Vol 2, pp. 829-841.

Engelhardt, K. & Golding, H. C. (1975). "Field Testing to Evaluate Stone Column performance in a Seismic Area " *Géotechnique*, Vol. **25**(1), pp. 61-69.

Enoki, M., Yagi, N. & Yatabe, R. (1991). "Evaluation of Bearing Capacity Analysis Method of Improved Ground." *Geo-Coast*, Yokohama, pp. 305-310

Hausmann, M.R. (1990). Engineering Principles of Ground Modification, 1st Edition, McGraw-Hill Publishing Co., p. 632.

Hughes, J. M. O., Withers, N. J. & Greenwood, D. A. (1975). "A Field Trial of the Reinforcing Effect of a Stone Column in soil." *Géotechnique*, Vol.**25**(1), pp. 31-34.

I.S.: 2720-V. (1965). "*Determination of Liquid & Plastic Limits*" ISI, New Delhi.

Ketkar, D. J. & Telang, (1994). "Comparative Performance of Petroleum Tannks on Ground Treated by Sandwicks, Stone Piles & Stone Columns" *Proc XIIIth Int Conf. on Soil Mechanics & Foundation Engineering*, New Delhi. pp 1207-1210

Lee, Y. N. (1994). "Soil Improvement Work by Vibroflotation." *Proc Conf. on Deep Foundations & Ground Improvement Schemes*. Bangkok, Thiland. Vol.**2**, pp. 289-340.

Leung, C. F. & Tan, T. S.(1993) "Load Distribution in Soft Clay Reinforced by Sand Columns." *Int. Conf. on Soft Soil Engg.* (Recent Advances in Soft Soil Engineering), Guangzhou, China, pp. 779-784.

McManus, K. J. & Kulhawy, F. H. (1993). "Prepartation of Large-Size Laboratoray Deposits of Cohesive Soils." *Geotechnical Testing Journal*, Vol 16. No. **3**, pp. 372–382.

Madhav, M. R. & Vitkar, P. P. (1978). "Strip Footing on Weak Clay stabilized with Granualr Trench or Pile." *Can. Geotech. Journal*, Vol **15**(4), pp 605-609.

Madhav, M. R. (1982). "Recent Developements in Use and Analysis of Granular Piles." *Proc. Symp. on Recent Developements in Ground Improvement Techniques*, Bangkok, pp. 117-129.

Madhav, M. R. & Thiruselvem, C. (1988). "Effect of Installation Methods on Granular Pile Response." *Proc. Int. Conf. on Theory and Practice of Earth Reinforcement*, IS-Kyushu, Fukuoka, pp. 221-226.

Madhav, M. R. (1989). "Bearing Capacity and Settlement Analysis of Granular Piled Raft." *Proc. 55 th R & D Session*, CBIP, Srinagar.

Madhav, M. R. (1992). "Shear Resistance of Granular Pile Reinforced Soil." *Proc. Int. Symp. on Soil Improvement and Pile Foundation* (ISSIPF '92), Nanjing, China, pp. 298-303.

Madhav, M. R. & Miura, N. (1994). "Stone Columns". Panel Report, *Proc. XIIIth Int. Conf. on Soil Mechanics & Foundation Engineering*, New Delhi, pp. 163-164.

Madhav, M. R. (1994). "Granular Plies - Recent Contributions." *Proc Workshop on Ground Improvement with Geotextiles & Stone Columns*, Calcutta

Matsuo, K. (1995). "Lessons from Kobe Earthquake." *Civil Engineering*, ASCE, Vol 64, No. 4.

Mitchell, J. K. (1981) "Soil Improvement - State-of-the-Art Report " *Proc. Xth Int. Conf. on Soil Mechanics & Foundation Engineering*. Stockholm, Vol 4, pp. 509-565.

Mitchell, J. K. & Wentz, F. J. (1991) "Performance of Improved Ground During the Loma Prieta Earthquake." UCB/EERC Report 91/92

Nagpure, D. D. & Madhav, M. R (1994). "Stability of Embankments on Soft Soils Reinforced by Granular Piles " *Proc. 74th AGM*, Inst. of Engrs (India), U P State Center, Lucknow, pp 18-22.

Nayak, N. V. (1982). "Recent Innvotations in Ground Improvement by Stone Columns." *Proc Symp. on Recent Developements in Ground Improvement Techniques*, Bangkok, pp. 17-29.

Priebe, H. (1978). "Abschatzung des Setzungsverhaltens eines durch Stopfverdichtung verbeserten Baugrundes." *Die Bautechnik*,**53**, H 5, S. 160-162, 2-4, Translation - *Design criteria for Ground Improvement by Stone Columns*

Priebe, H. (1988). "Zur Abschatzung des Setzungsverhaltens eines durch Stopfverdichtung verbeserten Baugrundes." *Die Bautechnik*,**65**, H 1, S. 23-26. Translation - *On assessing Settlement behaviour of a Soil Improved by Vibro-replacement.*

Randolph, M. F. (1994). "Design Methods for Pile Groups and Piled Rafts." *Proc. XIIIth Int. Conf. on Soil Mechanics & Foundation Engineering*, New Delhi, pp 61-82.

Ranjan, G. (1989). "Ground Treated with Granular Piles and its Response under Load." *Indian Geotechnical Journal*, Vol. **19**, No. 1, pp 1-85.

Rathgeb, E. & Kutzner, C.(1975). "Some Application of the Vibro-replacement Process." *Géotechnique*, **25**(1), pp. 45-50.

Sarma, U. V. (1994). "Interpretation of In-situ Test Data for two sites improved by Vibroflotation." *M. Tech. Thesis*, I.I.T., Kanpur, India.

Sabhahit, N. (1994). "Stability Analysis of Reinforced Problems - A Nonlinear Programming Approach." *Ph. D Thesis*, I.I.T., Kanpur, India.

Saito, A. *et al.* (1986). "A Countermeasure for Sand Liquefaction – 'Gravel Drains Method'." Nippon Kokan Technical Report, Vol **52**, pp. 46-52.

Van Impe, W. F. & de-Beer, E. (1983) "Improvement of Settlement Behaviour of Soft Layers by means of Stone Columns" *Proc 7 th ECSMFE*, Helsinki, Vol. 1, pp. 1207-1210.

Van Impe, W. F. & Madhav M R. (1992) "Analysis and Settlement of Dilating Stone Column Reinforced Soil". *Osterreichische Ingenieur and Architeckten-Zeistchrift (OIAZ)*, 137, Heft 3, pp 114-121.

Wood, D. M. (1985). "Some Fall Cone Tests". Technical Note. *Géotechnique*, Vol. **35**, (1), pp. 64–68

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